# JOURNAL
OF
# AMERICAN CONCRETE INSTITUTE

VOL. III FEBRUARY, 1915 No. 2


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## PURPOSE OF THE INSTITUTE.

It is the purpose of the Institute, by means of conventions and publications, to increase and disseminate knowledge in regard to the use of cement, concrete and allied products; to conduct research into their properties and uses; and to formulate recommended practice and standard specifications.

Its objects are purely educational.

* In place of B. F. Affleck, resigned.
† In place of W. L. Church resigned.

# JOURNAL
OF
# AMERICAN CONCRETE INSTITUTE

FOUNDED 1905, AND INCORPORATED 1906, AS
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VOL. III FEBRUARY, 1915 NO. 2


## ELEVENTH ANNUAL CONVENTION

The Eleventh Annual Convention of the Institute will be held at the Auditorium Hotel, Chicago, Ill., February 9–12, 1915. This Convention will mark the completion of the tenth year of the existence of the Institute, and an especially interesting and profitable program has been arranged. The sessions will be at 10 A. M., 3 P. M. and 8 P. M. The following is a summary of the program giving the subject discussed each day:

| | |
|---|---|
| **Tuesday, February 9, 1915.** | **Concrete Roads and Bridges.** |
| **Wednesday, February 10, 1915.** | **Concrete and Reinforced Concrete Tests and Design.** |
| **Thursday, February 11, 1915.** | **Concrete in Art and Architecture.** |
| **Friday, February 12, 1915.** | **Plant Management and Cost. Banquet.** |

**Tenth Anniversary and Lincoln Birthday Banquet Friday Evening**

## Membership Certificate.

A Certificate of Membership in this Institute will be furnished to any member in good standing, i. e., whose dues are paid up to date. These Certificates are printed from an engraved steel plate. The name of the member is engrossed and the Certificate will be signed by the President and Secretary. The Certificates are about 10 x 14 ins., and have sufficient margin to be framed 14 x 17 ins. They are engraved on parchment or parchment paper, both suitable for rolling or framing.

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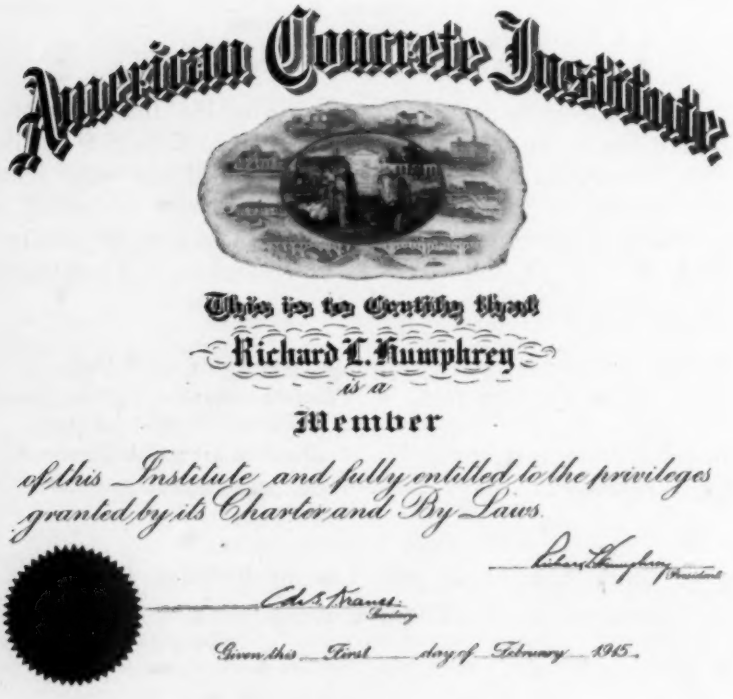
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Richard L. Humphrey
is a
Member
of this Institute and fully entitled to the privileges granted by its Charter and By Laws.
Given this First day of February 1915.

MEMBERSHIP CERTIFICATE.

## Standards Adopted by the Institute.

1. Standard Specifications for Cement with Appendix on Standard Methods of Testing and Chemical Analysis, 23 pp.
2. Standard Specifications for Portland Cement Sidewalks, 6 pp.
4. Standard Building Regulations for the Use of Reinforced Concrete, 13 pp.
5. Standard Specifications for One Course Concrete Highway, 10 pp.
6. Standard Specifications for Portland Cement Curb and Gutter, 7 pp.
7. Standard Recommended Practice for the Use of Reinforced Concrete, 21 pp.
8. Standard Specifications for Scrubbed Concrete Surfaces, 3 pp.
9. Standard Recommended Practice for Concrete Drain Tile, 3 pp.
10. Standard Recommended Practice for Concrete Architectural Stone, Building Block and Brick, 4 pp.
11. Standard Specifications for Concrete Architectural Stone, Building Block and Brick, 3 pp.
12. Standard Building Regulations for the Use of Concrete Architectural Stone, Building Block and Brick, 3 pp.
13. Standard Specifications for Plain Concrete Floors, 6 pp.
14. Standard Specifications for Reinforced Concrete Floors, 5 pp.
15. Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Block, 9 pp.
16. Standard Methods for the Measurement of Concrete Work, 7 pp.
17. Standard Specifications for One Course Concrete Street Pavement, 10 pp.
18. Standard Specifications for Two Course Concrete Street Pavement, 11 pp.

The above Standards can be obtained at the price of 25 cents each or in lots of ten or more at 20 cents each. Special price to Members 15 cents each or 10 cents each in lots of ten.

## Synopses of Recent Articles on Concrete.

**Reviewed January 1 to 31, 1915.**

The synopses given below are for the purpose of keeping the members of the Institute in touch with the leading articles of interest appearing in the many technical journals and is not a complete review of the journal. The publications regularly reviewed appear below and the number given corresponds to the number following the title of the paper. The length of the article in pages, whether illustrated or not, is noted, and the publications reviewed in which it appears:

1. Canadian Engineer, Toronto, Ontario, Canada.
2. Cement, New York, N. Y.
3. Cement Era, Chicago, Ill.
4. Cement World, Chicago, Ill.
5. Concrete Age, Atlanta, Ga.
6. Concrete Era, Los Angeles, Cal.
7. Concrete-Cement Age, Detroit, Mich.
8. Concrete and Constructional Engineering, London, England.
9. Engineering and Contracting, Chicago, Ill.
10. Engineering News, New York, N. Y.
11. Engineering Record, New York, N. Y.
12. Good Roads Magazine, New York, N. Y.
13. Portland Cement, Kansas City, Mo.
14. Rock Products, Chicago, Ill.
15. Proceedings American Railway Engineering Association, Chicago, Ill.
16. Western Contractor, Record Building, Kansas City, Mo.
17. Journal National Fire Protection Association, Boston, Mass.
18. Journal Western Society of Engineers, Chicago, Ill.
19. Proceedings American Society of Civil Engineers, New York, N. Y.
20. Proceedings American Society of Municipal Improvements, New York.
21. Proceedings American Society for Testing Materials, Philadelphia, Pa.
22. Proceedings Engineers' Club of Philadelphia, Philadelphia, Pa.
23. Proceedings Engineers' Society of Western Pennsylvania, Pittsburgh, Pa.
24. Professional Memoirs, Corps of Engrs., U. S. A., Washington, D. C.
25. Proceedings Institution of Civil Engineers, London, England.
26. Transactions, Canadian Society of Civil Engineers, Montreal, Quebec, Canada.
27. Railway Age Gazette, Chicago, Ill.
28. Better Roads, Jamestown, Ohio.

## MATERIALS

**Field Methods in Concrete Construction—A Consideration of Materials,** by Jerome Cochran. (7) Jan., 2 pp. An article on concrete materials; will be followed by others on proportioning and mixing, forms and centering, etc.

**The Bulk Handling of Cement,** by H. M. Capron. (3) Jan., 2 pp.

## DESIGN AND TESTS

**Design Methods in Concrete Construction—Analytical Statics of Simple Beams,** by Jerome Cochran. (7) Jan., 2 pp., ill. Mathematical treatise.

**Tests for Bond and Electrolytic Corrosion of Painted Reinforcing Steel.** (10) Jan. 21, 1¾ pp., ill.

**Problems in the Theory of Construction—Column Bending Moments in Frame Construction,** by Ewart S. Andrews. (8) Jan., 7 pp., ill.

**The Action of Boiling Water on Concrete,** by E. R. Matthews. (8) Jan., 3 pp., ill.

**Design and Structural Details of Concrete Lining for Irrigation Canals.** (9) Jan. 27, 3 pp., ill.

**Test of Full-Size Reinforced-Concrete Bridge Slab.** (11) Jan. 2, 1 p., ill. Determination at Arlington Experimental Farm of stress distribution and other phenomena in slab under various loadings. Slab 16 ft. clear span, 32 ft. wide and 12 in. deep.

**Reinforced-Concrete Highway Trestle,** by S. M. Cotton. (11) Jan. 9, ¾ p., ill. Description of loadings, distribution for maximum stress and design of standard structure for the State of Arizona.

**Routine Tests for Determining Strength of Mortars,** by Harold Perrine. (11) Jan. 16, ¾ p., ill. Description of method involving use of diagonally mounted mixing box and 2 x 4-in. cylindrical test pieces.

**Microscope Opens New Field in Study of Concrete,** by Nathan C. Johnson. (11) Jan. 23, 4½ pp., ill. The first of a series of six articles treating of the causes of such defects as exist in concrete and indicating remedies therefor.

## BUILDINGS

**The Fire-Resisting Qualities of Concrete Buildings at the Edison Factory, West Orange, N. J.** (7) Jan., 1¾ pp., ill. Plan showing location and type of buildings, with brief description of the fire and its effect on concrete construction.

**Unit Construction in Concrete Mill Building.** (1) Jan. 21, 3 pp., ill.

**Coaling and Sanding Station on the Virginian Railway at Elmore,** by F. F. Harrington. (11) Jan. 2, 2 pp., ill. Reinforced-concrete structure with crusher and screen, accommodating 200 tons each of lump and stoker coal and 150 tons of green sand.

**Restoring Concrete Buildings at Edison Plant.** (11) Jan. 9, ½ p., ill. Sections of columns, beams and girders are being increased and new reinforcement added. Description of work.

**Restoring Concrete Buildings at Edison Plant.** (11) Jan. 16, ½ p., ill.

**Preliminary Repair Work on the Edison Concrete Buildings.** (10) Jan. 14, 2 pp., ill. Description of work.

**Paper-Covered Frames for Cold Weather Concreting,** by A. A. Lane. (11) Jan. 9, ½ p., ill. Protection costing 2.1 c. per sq. ft. placed to enclose 46 x 212-ft. two-story building in Ohio during construction.

## HOMES

**Houses at Forest Hills Gardens—Pre-cast Hollow Concrete Floor, Wall and Roof Units and Exposed Aggregates,** by Frederick Squires. (7) Jan., 9 pp., ill. Description and views of town on Long Island showing the variety and extensive use of concrete in home and building construction; also products factory in connection with building the town.

**Stucco in Suburban Architecture—Notes on Ogontz Hill, Philadelphia,** by Oswald C. Hering. (7) Jan., 6 pp., ill. Artistic homes of hollow tile stuccoed, including half timbers.

**Concrete Block Used in Attractive and Economical House Construction at Mooseheart, Ill.** (7) Jan., 2 pp., ill. Description and plan of two-story building 33 x 33 ft.

**Some "Worked Out" Details in Concrete House Construction.** (7) Jan., 1¼ pp., ill. Elimination of material liable to rust or rot, by the use of concrete.

**Two Cottages with Solid Concrete Walls, Built near Middleburg, Va.** Description and plans of two cottages costing $2,450 and $2,700, respectively.

**An Engineer's Concrete Block Cottage—Built in Riverside, Cal., for $3,000.** (7) Jan., 1⅔ pp., ill. Plan, description and detailed construction cost and performance of work.

**House with Concrete Walls and Permanent Interior "Form" of Hollow Tile,** by C. R. Knapp. (7) Jan., 1¾ pp., ill. Description of the use of a movable core system in wall construction.

**Thin Stucco Finish on Garden Walls and Buildings Near Middleburg, Va.** (7) Jan., 1½ pp., ill. Stucco ¼ in. average thickness, cost 15 cents per sq. yd. for labor and material.

**Suggestions for Light Concrete Floor Construction in the Concrete House,** by Milton Dana Morrill. (7) Jan., 1 p., ill.

**Country Home of Reinforced Concrete,** by Oliver Randolph Parry. (3) Jan., 3 pp., ill. Description and views of handsome concrete residence.

## BRIDGES AND RAILROAD WORK

**Two Miles of Concrete Trestles.** (3) Jan., 2½ pp., ill. Description of pre-molded units for railroad trestles.

**Railroads Building Concrete Bridges.** (3) Jan., 2 pp. Extent reinforced concrete is in use for bridges, the types developed and in most general use, and the practice of different railroads regarding standard types and specifications for design.

**What the B. & O. has Done with Concrete.** (3) Jan., 2 pp. A continuation from November issue.

**The 12th St. Double-Deck Viaduct at Kansas City.** (10) Jan. 7, 5 pp., ill. Details of design and construction of reinforced concrete structure 2265 ft. long, having max. height of 110 ft. across the bottom lands of Kansas River, concrete-pile founded.

**The Columbia Highway in Oregon,** by Henry L. Bowlby. (10) Jan. 14, 2½ pp., ill. Description and views of reinforced concrete viaduct 860 ft. long and two arch concrete bridges.

**A Bridge Sidewalk Failure Due to Paving Expansion.** (10) Jan. 14, 1 p., ill. Reinforced concrete sidewalk cantilever of Dallas, Tex., viaduct failure due to expansion of wood-block pavement.

**Important Questions on Highway-Bridge Design,** by Willis Whited. (10) Jan. 21, 1½ pp.

**St. Paul and Oregon-Washington Joint Terminals in Spokane.** (27) Jan. 15, 4 pp., ill. Showing types and method of constructing street subways.

**Lining the Snoqualmie Tunnel.** (27) Jan. 22, ¾ p., ill. Showing method of placing concrete arch lining before removing bench excavation.

**New I. C. Station and Track Elevation at Memphis.** (27) Jan. 29, 2¾ pp., ill. Interesting use of concrete described.

**Data on Reinforced Concrete Railroad Bridges and Extent to which such Bridges are Used.** (9) Jan. 13, 3 pp., ill.

**Effect of Duration of Mixing on the Strength of Concrete.** (9) Jan. 27, 1½ pp.

**Substructure of the Trent River Bridge.** (11) Jan. 9, ¾ p., ill. Description of concrete pedestals for steel towers and wooden abutments built to avoid retaining walls in fresh banks.

**Brooklyn Bridge Subway Connection, New York City.** (11) Jan. 16, 1 p., ill. Description of difficult removal of large quantities of massive concrete for construction of the inclined tracks in Manhattan Terminal.

**Causeway at New Philadelphia, Ohio,** by Edward Stingel. (11) Jan. 16, 1 p., ill. Design, construction and special features of reinforced concrete bridge replacing steel bridge washed out by 1913 flood.

## RESERVOIRS, DAMS, CANALS, ETC.

**Lining the St. Louis Water Tunnel with Concrete by Means of Compressed Air,** by E. C. Davis. (10) Jan. 28, 2¾ pp., ill. Description of.

**Construction of Platt Avenue Siphon.** (3) Jan., 1¼ pp., ill. Description of reinforced concrete siphon 168 ft. long, internal diameter 15 ft. 7 in.

**Dam and Log Slide, High Falls, Que.,** by Jos. Percy McRae. (1) Jan. 7, 3½ pp., ill. Details and description of.

**Proposed Method of Constructing Pearl Harbor Dry Dock.** (10) Jan. 14, 3¾ pp., ill. Details of scheme to replace dock in Hawaii blown up in February, 1913.

**The City Tunnel of the Catskill Aqueduct,** by Walter E. Spear. (10) Jan. 21, 5¾ pp., ill. II.—Construction of shafts.

**Hydro-Electric Power Development at Cedar Rapids, Que.** (1) Jan. 21, 2¼ pp., ill. Description and construction methods.

**Puntledge River Power Development.** (1) Jan. 28, 4 pp., ill. Description of the various uses of concrete.

**Reinforced Concrete Power House, New Zealand,** by H. C. Gayford. (8) Jan., 5½ pp., ill. Views and description.

**Rio Grande Reclamation Project,** by W. D. Hornaday. (14) Jan. 22, 1 p., ill. Description of huge Elephant Butte Dam.

**Construction of Cumberland Waterworks,** by Frank H. Eastman. (11) Jan. 30, 2¾ pp., ill. Hauling problems, layout of dam site, including quarry and cement-block plant, and excavation for nine-mile wood-stave conduit.

**New Plan for Building Pearl Harbor Drydock near Honolulu.** (11) Jan. 16, 2 pp., ill. Large naval structure on which the cofferdam method was found unsafe will be constructed in full-width sections 60 ft. long, of reinforced-concrete construction.

## ROADS AND PAVEMENTS

**Analysis of Concrete Curb Construction for Suburban Improvements near New York City.** (9) Jan. 20, 3 pp., ill.

**Curing Wisconsin Concrete Road Under Water.** (11) Jan. 2, ¼ p., ill. Brief description of flooding green concrete 10 to 14 days where grade does not exceed 2 per cent.

**Costs Reduced by Monolithic Curb, Gutter and Pavement.** (11) Jan. 23, 1 p., ill. Economies effected in Middle West by integral curb construction which added only few cents per yard to pavement cost.

## SEWERS, PIPE TILE, ETC.

**Concrete Sewer Pipe Specifications,** by E. S. Hanson. (3) Jan., 2½ pp. A discussion, to be continued in succeeding issues.

**Cleveland West Side Water-Supply Tunnel.** (10) Jan. 7, 4¼ pp., ill. Details and description of construction of a 10-ft. diam., 3-mile tunnel now being driven under Lake Erie from existing crib 1½ miles offshore to new crib nearly 4 miles offshore, lined with concrete blocks 18 in. deep and 11 in. thick placed six to the circumference.

**New Methods of Pneumatic Tunneling Aid Safe and Rapid Completion of Passaic Valley Sewer Contract.** (11) Jan. 30, 3½ pp., ill. Departure from usual heading practice, and use of light, pressed steel lining plates, expedite driving of 12-ft. concrete-lined bore under Newark streets.

## CONCRETE PRODUCTS

**Concrete for Ornamental Garden Work.** (8) Jan., 5½ pp., ill.

## MISCELLANEOUS

**Unique Features in a Municipal Pier.** (3) Jan., 2 pp., ill. Entire substructure and deck of reinforced concrete, including interlocking concrete sheet piling, long concrete bearing piles and shore anchorages of concrete.

**Concrete Good for Small Grandstand,** by Raymond Thompson. (3) Jan., 1¾ pp., ill. Description and details of small grandstand.

**Tearing Down a 250-ft. Reinforced Concrete Chimney.** (10) Jan. 14, 1 p., ill. Description and cost of razing by piecemeal on account of proximity of adjacent building.

**Reinforced Concrete Race-Course Stand, Cheltenham,** by Albert Lakeman. (8) Jan., 7 pp., ill. Details and description of.

**Deep Water Concreting Through a Tube,** by E. B. Van De Greyn. (8) Jan., 4 pp., ill. Notes on the Tremie method and suggestions of various expedients to avoid the difficulties likely to be encountered.

**Concrete on the Farm,** by Dr. Leonard Keene Hirshberg. (14) Jan. 22, 1 p., ill.

**Grouting a Leaking Tunnel.** (11) Jan. 9, ½ p., ill. Description of work.

**Floating Mixing Plant with Tower for Concreting on River Work.** (11) Jan. 16, ½ p., ill. Constructed on hull 30 x 75 ft.; averaged 30 yd. per hour run of mix; average cost mixing and placing from 55 to 90 cents per yd.

To be presented at Chicago Convention—1915.
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# STRENGTH OF CONCRETE FORMS.

By H. S. Taft.*

## 1. Introduction.

Though concrete form-work is of a very temporary nature, it is of the utmost importance that it be designed and constructed to meet the loads brought upon it by the wet concrete; otherwise disaster is likely to result to the temporary structure. While at first form-work was more or less of a hit and miss, now that concrete construction has become so universal, the design and building of the form-structure is being studied and looked after almost as carefully as the concrete itself. Thus it is fitting to consider the question of strength of concrete forms and how best to determine the sizes and spacing of the various members that go to make up a form in toto.

## 2. Forms Treated as Beams.

Concrete forms in their final analysis are nothing more nor less than various types of beams uniformly loaded. When the points of support are all on the same straight line, the beams are in the nature of continuous girders. How far they depart from being continuous girders depends upon how far the points of support depart from a straight line. Instead of being treated as continuous girders, forms can be considered:

1. As beams fixed in direction or supported at both ends.
2. As beams supported at one end and fixed at the other.

In each case the spacing of the supports, etc., can be figured in two ways:

(*a*) For a maximum deflection.

(*b*) For a maximum extreme fibre stress.

In making the calculations the first thing to determine is the load carried by the form member in question. Such members as the sheathing, joists and girders of floor slab form, the bottom boards of beam forms, their supporting struts, etc., can be directly

*Contracting Engineer, Seattle, Wash.

calculated for deflection or strength from the weight of the wet concrete carried by the forms. The form-work of walls, footings, etc., cannot be so directly figured since the exact pressure upon the surface of such types of forms is not absolutely known.

Col. W. D. Douglass states that the pressure of wet concrete upon the forms for columns or walls of considerable size where the concrete is poured at one operation may be taken as of a liquid nature. He assumes wet concrete as a liquid of one-half its weight,

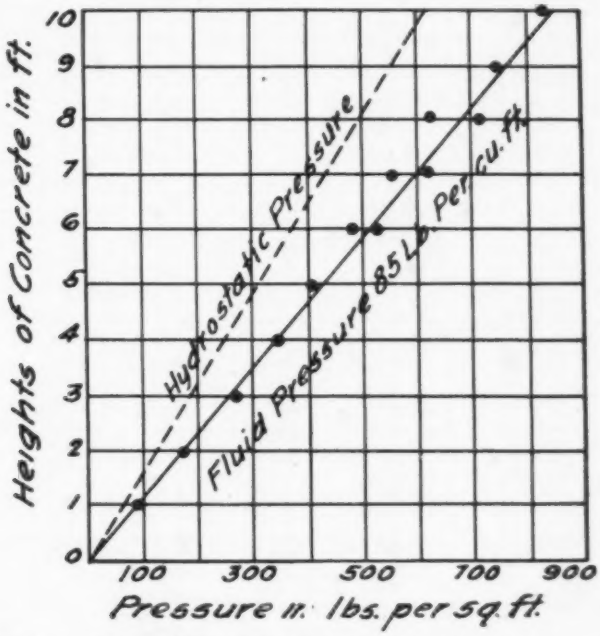



FIG. 1.—TESTS TO DETERMINE PRESSURE OF LIQUID CONCRETE, BY H. ST. G. ROBINSON.

that is a liquid weighing 75 lb. per cu. ft., and finds that this figure gives good and safe results. If it takes a long time to fill the forms, the lower layers of the concrete will have taken their set and become hardened long before the top layers have been poured, in which case this assumption would be erroneous but on the side of safety. On the other hand, if the forms were rapidly filled, the entire mass would be in such a liquid state as to make the results not only erroneous but on the side of danger.

### 3. PRESSURE TESTS.

During past years a number of experiments and tests have been made to determine the pressure upon form work due to wet concrete.

In a series of tests made by Hector St. George Robinson in England upon lateral pressure of liquid concrete, the lateral pressure exerted was found to be fairly uniform and practically constant for each head; but in reinforced concrete columns of

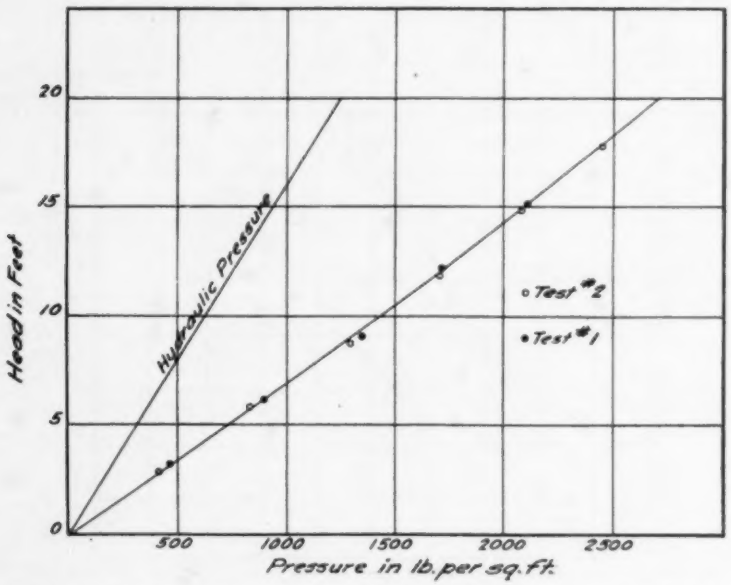



FIG. 2.—TESTS TO DETERMINE LATERAL PRESSURE OF WET CONCRETE, ABERTHAW CONSTRUCTION COMPANY.

small dimensions, thin walls and other light concrete work, the effect of friction between the more or less rough timber forms and the concrete, together with the arching action, was found to reduce the pressure considerable. The general conclusions drawn from these tests and other experiments was that the lateral pressure of concrete for average conditions is equivalent to that of a fluid weighing 85 lb. per cu. ft. When but little water is used and a very dry mixture is had, the pressure reduces to a liquid of 70 lb. per cu. ft.

Fig. 1 gives the results of some of the above tests. In the first series the wall was 3 ft. thick, the concrete of a 1-3-6 mixture, weighing 140 lb. per cu. ft., thoroughly plastic and carried up rapidly to 8 ft. above the center of face pressure. In the second series the form was a 4-ft. square pier, filled with a 1-2-4 mixture weighing 145 lb. per cu. ft. The same plastic mixture was used and carried up to 10 ft. above the center of face pressure.

Several tests were recently made by The Aberthaw Construction Company of Boston, to determine the probable pressure of

MAXIMUM PRESSURE OF CONCRETE ON FORM.

From Major F. R. Shunks' Experiments at Lock No. 1, Mississippi River, St. Paul

| Rate of Filling | | Temperature of the Concrete. | | | | |
|---|---|---|---|---|---|---|
| | | 80 | 70 | 60 | 50 | 40 |
| Vertical Rise in Feet Per Hour. | 2 | *3.7* 530 | *3.9* 560 | *4.3* 600 | *4.7* 680 | *5.5* 790 |
| | 3 | *4.8* 690 | *5.0* 720 | *5.6* 810 | *6.4* 920 | *7.5* 1080 |
| | 4 | *5.7* 820 | *6.0* 870 | *6.8* 980 | *7.9* 1130 | *9.3* 1340 |
| | 5 | *6.5* 930 | *6.9* 990 | *7.8* 1120 | *9.1* 1310 | *10.9* 1570 |
| | 6 | *7.1* 1020 | *7.6* 1090 | *8.7* 1250 | *10.3* 1480 | *12.4* 1780 |
| | 7 | *7.6* 1090 | *8.1* 1170 | *9.4* 1350 | *11.2* 1620 | *13.7* 1970 |
| | 8 | *7.9* 1130 | *8.6* 1240 | *10.0* 1440 | *12.1* 1740 | .... .... |

*Italic* figures are pressures in lb. per sq. in
Roman figures are pressures in lb. per sq. ft.
See Fig. 3 for plotted results.

wet concrete in an ordinary building column form. The tests were made upon a column form about 18 ft. in height, the time of pouring in one case being 9 minutes, in another 14 minutes. In both cases the pouring had been completed before any initial set had begun to take place at the base of the column; the whole mass of concrete thus being in a liquid state. These tests show that in the ordinary column, as found in buildings, it is necessary to consider the lateral pressure of wet concrete as that of a liquid weighing 140 to 150 lb. per cu. ft., and not 72 lb. as recommended

by Col. Douglass for large piers and columns, nor 85 lb. as determined from the above English tests. Fig. 2 gives the results of the Aberthaw tests.

The work of Major F. R. Shunks is perhaps the best of all data had upon the subject as respect to forms holding large masses; especially so as the original results of Major Shunks were checked up by still more extensive experiments at Panama and found to be correct.

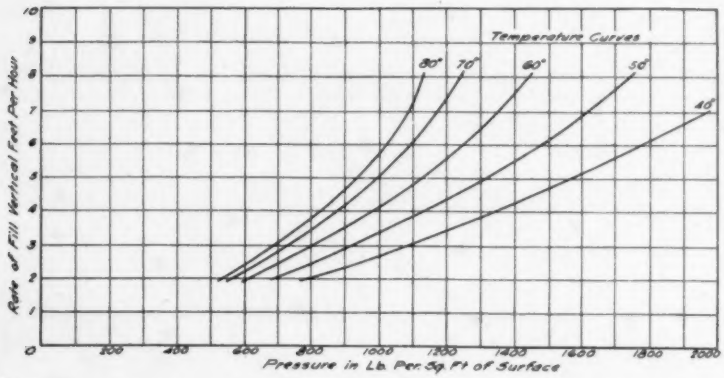



FIG. 3.—TESTS TO DETERMINE LATERAL PRESSURE OF CONCRETE, BY MAJOR F. R. SHUNKS.

## 4. GENERAL EQUATIONS.

In determining the strength of the various members of a form-structure it is necessary to figure them for both deflection and strength and then use the smaller of the two values, as will be more fully explained later on. The maximum deflection and bending moments for uniformly loaded beams, as expressed by well-known formulæ are:

| Type of Girder. | For Deflection in inches | For Strength. |
|---|---|---|
| 1. Continuous girder | ——— | $M = \frac{Wl}{10}$ |
| 2. Supported at both ends | $d = \frac{5}{384} \frac{Wl^3}{EI}$ | $M = \frac{Wl}{8}$ |
| 3. Fixed in direction at both ends | $d = \frac{1}{384} \frac{Wl^3}{EI}$ | $M = \frac{Wl}{12}$ |

Considering the real nature of form construction the following values have been adopted as fair averages of conditions met with in actual construction work, figures which are means between those for a beam supported at both ends, and a beam fixed in direction at both ends:

$$d \text{ (deflection in inches)} = \frac{3}{384}\,\frac{Wl^3}{El} = \frac{1}{128}\,\frac{Wl^3}{EI} \qquad \text{(A)}$$

and

$$M \text{ (bending moments)} = \frac{1}{10}\,Wl = \frac{1}{6}\,bt^2 f$$

$$\therefore Wl = \frac{5}{3}\,bt^2 f \qquad \text{(B)}$$

Wherein $b$ equals the width and $t$ the thickness of the form member, viz., the lumber; $W$ the total load upon the form member between supports; and $l$ the distance between supports.

## 5. WEIGHT OF CONCRETE PER CUBIC FOOT; LIVE LOAD PER SQUARE FOOT; LOAD ON STRUTS.

In figuring the weight of wet concrete some engineers work on a basis of 150 lb. per cu. ft. though it is claimed that 154 lb. is more accurate for reinforced concrete (making approximately 100 lb. of steel per cu. yd. of concrete). Others use a figure of 144 lb. per cu. ft. since it greatly simplifies the work of computation. When the factor of 144 lb. per cu. ft. is used the area of the cross section in square inches of the member in question equals its weight in lb. per lineal foot. A short method of making weight calculations.

To insure safety to the temporary form structure allowances must be made for a live load upon its various members due to construction material, tools, etc. The following allowances will be found to be well within common practice:

For floor beams.....50 lb. per sq. ft.
For floor slabs, stringers and struts.....75 " " " "

If, instead of using these exact quantities, loads of 48 lb.and 72 lb. for beam and floor slabs, respectively, are used, it still further simplifies the calculations without any material change in

the results, (48 being divisible by 12 and 72 being a divisor of 144).

The spacing of struts under beam and floor slab forms must not only be such as to keep the deflection and strength of the various members within bounds of safety, but it must also be determined whether such spacing is excessive for the loads thrown thereby upon the supporting struts. A compression strength of 600 to 1200 lb. per sq. in. can be taken with safety for the struts, depending upon their length. The following table gives safe allowance loads for different size struts and different lengths as recommended by a leading authority upon concrete work.

LOAD ON STRUTS

| Unbraced Length of Strut, in ft. | Size of Struts. | | | |
|---|---|---|---|---|
| | 3 x 4 | 4 x 4 | 6 x 6 | 8 x 8 |
| 6 | *1,000*<br>12,000 | *1,200*<br>19,200 | *1,200*<br>43,200 | *1,200*<br>76,800 |
| 8 | *850*<br>10,200 | *1,050*<br>16,800 | *1,200*<br>43,200 | *1,200*<br>76,800 |
| 10 | *700*<br>8,400 | *900*<br>14,400 | *1,100*<br>39,600 | *1,200*<br>76,800 |
| 12 | *600*<br>7,200 | *800*<br>12,800 | *1,000*<br>36,000 | *1,200*<br>76,800 |
| 14 | ....<br>.... | *700*<br>11,200 | *900*<br>32,400 | *1,100*<br>70,400 |

*Italic* figures = safety load, lb. per sq. in.
Roman figures = total safety load, lb. per strut.

## 6. HYDROSTATIC AND LOAD PRESSURE. CONSTANTS.

The nature of the pressure or load per unit upon concrete forms depends upon the type of the form. In columns and wall forms a hydrostatic condition exists. In slab work a direct load has to be considered. In beam forms a direct load as well as a hydrostatic pressure exists. In equations (A) and (B) the load or pressure per unit which the form in question has to carry becomes the all important factor. When this factor has been determined, all sizes for a form structure for building purposes can be determined from these two equations. For $E$ and $f$, values of 1,300,000 and 1,200 respectively have been adopted.

## 7. COLUMNS.

The hydrostatic pressure of the liquid concrete upon a column form is directly proportionate to its height, the pressure at any point being determined from the expression:

$$p \text{ (pressure per sq. in.)} = \frac{Wt \text{ of concrete per cu. ft. } h \text{ (height)}}{144}$$

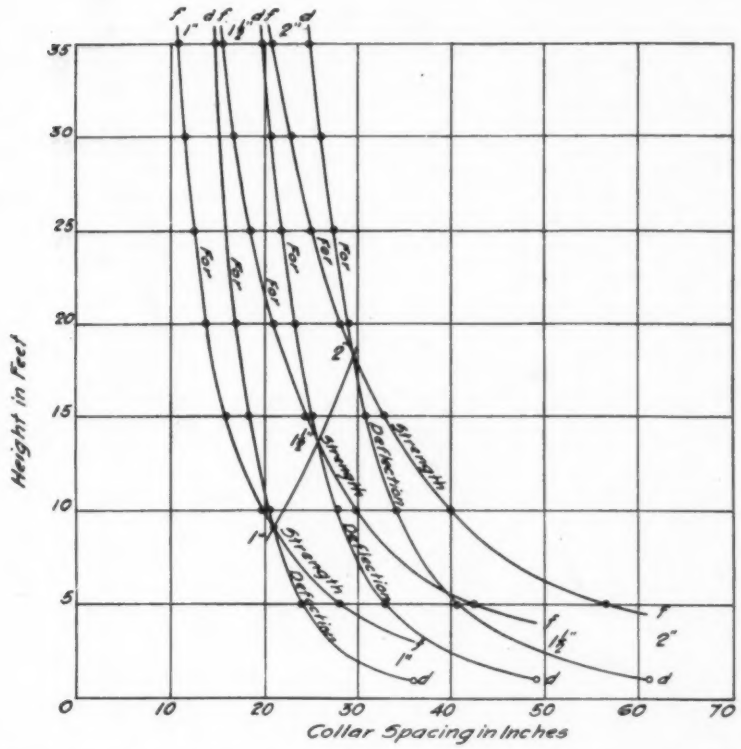



FIG. 4.—SPACING OF COLUMN COLLARS. WEIGHT OF WET CONCRETE = 72 LB. PER CU. FT. SEE TABLE I.

Adopting factors of 72 and 144 lb. for the weight of wet concrete per cu. ft., the expression becomes:

$p = \frac{1}{2} h$, for large piers;
$p = h$ for small piers or columns,

where $h$ is in feet. If $l$ equals the spacing of the collars, then the total load $W$ carried by a strip 1 in. wide, 1 in. long, will be:

$W = pl$ or $\frac{1}{2}\ l\ h$ for large piers.
$W = pl$ or $lh$ for small piers or columns.

A value of $\frac{1}{16}$ of an inch has been assumed for deflection, and thickness of 1, $1\frac{1}{2}$ and 2 in. taken for the form lumber. Substitut-

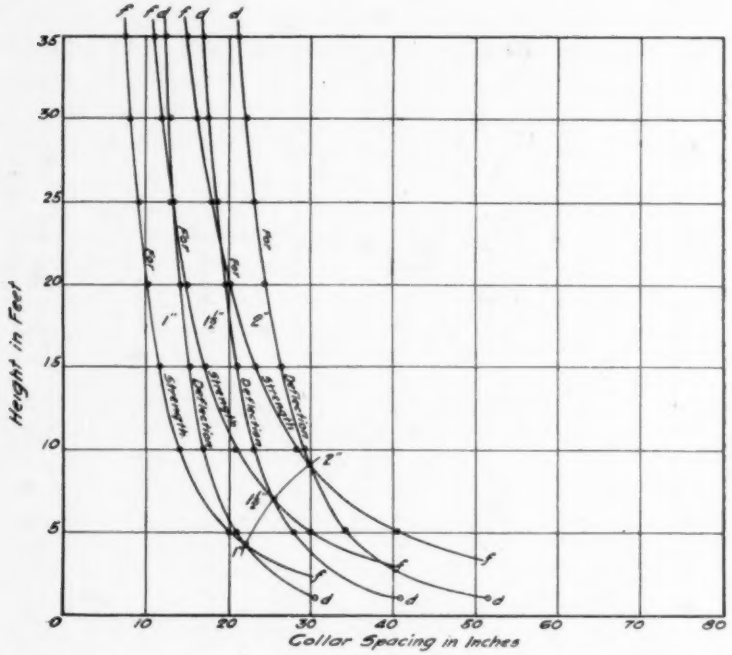



FIG. 5.—SPACING OF COLUMN COLLARS. WEIGHT OF WET CONCRETE = 144 LB. PER CU. FT. SEE TABLE II.

ing this data in equations (A) and (B), sets of formulæ have been deducted for determining the proper spacing of column collars. (See Appendix.—1. Columns.) The results of solving the equations for a height up to 40 ft. are given in Tables I and II. The same data is shown in forms of curves in Fig. 4 and 5.

On Fig. 6 and 7 three spacing curves are recorded. These curves are a combination of the lower part of the deflection curves and the upper part of the strength curves of Fig. 4 and 5.

From what has been said, it is evident that if collar or stud spacings are graduated from the upper part of the form to the lower so that in no case any one spacing exceeds that shown by these curves, such spacings will be within bounds of safety. Thus a broken line

TABLE I.—COLUMNS. BASED UPON A LIQUID WEIGHT OF 72 LB. PER CU. FT. USE FIGURES IN ITALIC.

| Height, feet. | Pressure, lb. per sq. in. | Spacing of Collars, inches. Thickness of Sheathing, inches. For Deflection. 1 | 1½ | 2 | For Strength. 1 | 1½ | 2 |
|---|---|---|---|---|---|---|---|
| 1 | .5 | *36* | *49* | *61* | 63 | 95 | 127 |
| 5 | 2.5 | *24* | *33* | *41* | 28 | 43 | 57 |
| 10 | 5.0 | 20½ | *28* | *35* | *20* | 30 | 40 |
| 15 | 7.5 | 18½ | 25 | *31* | *16* | *24½* | 33 |
| 20 | 10.0 | 17 | 23 | 29 | *14* | *21½* | *28* |
| 25 | 12.5 | 16 | 22 | 27 | *13* | *19* | *25* |
| 30 | 15.0 | 15½ | 21 | 26 | *12* | *17* | *23* |
| 35 | 17.5 | 15 | 20 | 25 | *11* | *16* | *21* |
| 40 | 20.0 | 14½ | 19½ | 24 | *10* | *15* | *20* |

TABLE II.—COLUMNS. BASED UPON A LIQUID WEIGHT OF 144 LB. PER CU. FT. USE FIGURES IN ITALIC.

| Height, feet. | Pressure, lb. per sq. in. | Spacing of Collars, inches. Thickness of Sheathing, inches. For Deflection. 1 | 1½ | 2 | For Strength. 1 | 1½ | 2 |
|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 30½ | *41* | *51½* | 45 | 67 | 90 |
| 5 | 5.0 | 20½ | *27½* | *34* | *20* | 30 | 40 |
| 10 | 10.0 | 17 | 23 | 29 | *14* | *21* | *28* |
| 15 | 15.0 | 15½ | 21 | 26 | *11½* | *17* | *23* |
| 20 | 20.0 | 14½ | 19½ | 24 | *10* | *15* | *20* |
| 25 | 25.0 | 13½ | 18½ | 23 | *9* | *13½* | *18* |
| 30 | 30.0 | 13 | 17½ | 22 | *8* | *12* | *16* |
| 35 | 35.0 | 12½ | 17 | 21 | 7½ | *11½* | *15* |
| 40 | 40.0 | 12 | 16½ | 20½ | *7* | *11* | *14* |

made up of a series of steps as shown to the left of the spacing curves, will give a means of determining a set of consecutive spacings that will closely follow the theoretical curves. Each step in the broken lines represents a change in spacing; changing quite rapidly at the upper part of the column or form, but more

gradually as the height increases, with several repetitions of the spacings for the greater depths. To the left is recorded in tabular form a series of continuous spacings of column collars and wall studs for 1, 1½ and 2-in. lumber; gradually decreasing as the height increases, all closely following the theoretical spacing.

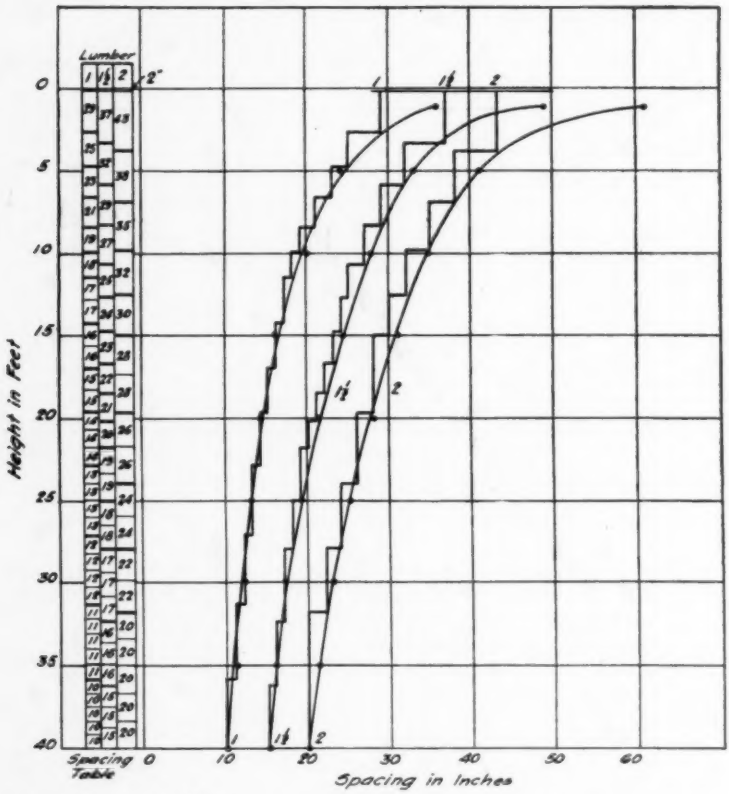



FIG. 6.—SPACING OF COLUMN COLLARS AND WALL FORM STUDS. WEIGHT OF WET CONCRETE 72 LB. PER CU. FT. CURVED LINES PLOTTED FROM TABLE I. STEPPED LINES REPRESENT SPACINGS.

In Fig. 8 an attempt has been made to compare the author's results with those of a well known authority upon concrete. The dotted lines represent the author's spacings; the others are recorded in full lines. While the spacings at the beginning and

end of the series are about the same, those of the author have a more gradual change as the height increases. To the right are recorded the spacings of both sets for three sizes of lumber.

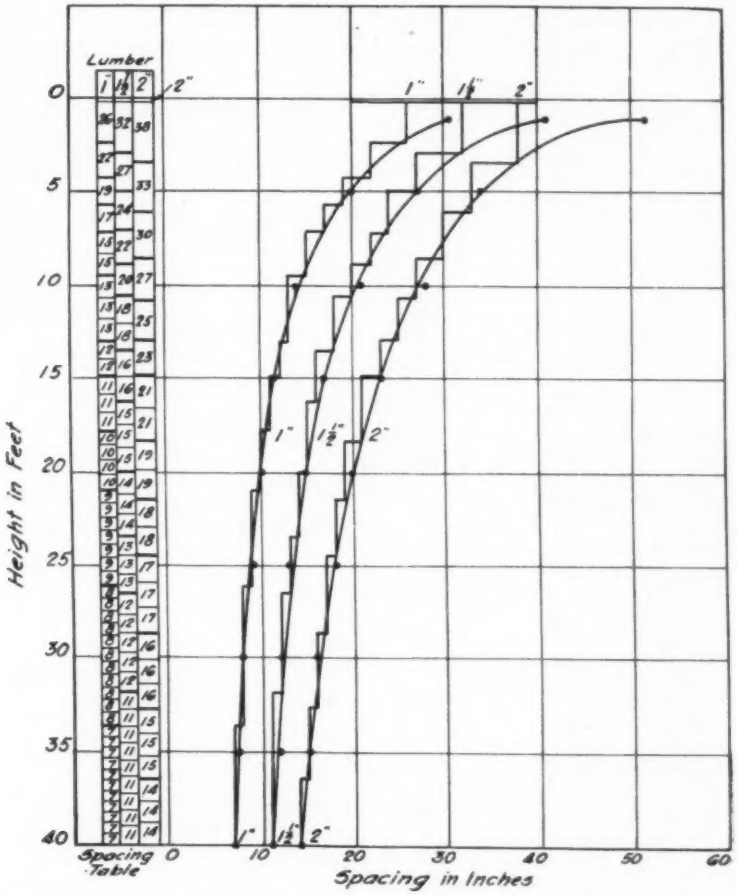



FIG. 7.—SPACING OF COLUMN COLLARS AND WALL FORM STUDS. WEIGHT OF WET CONCRETE 144 LB. PER CU. FT. CURVED LINES PLOTTED FROM TABLE II. STEPPED LINES REPRESENT SPACING.

It is evident that a 2 x 4-in. collar cannot be used above a certain size column, as it would not be strong enough to resist bending due to the load brought upon it. A column collar is under

very much the same condition as a stud to a wall form. Thus the results of the calculations for columns and wall forms are interchangeable. The maximum size of columns the various collar sizes noted are suitable for can be determined from Table III. Knowing the height and the thickness of the sheathing, the figures at the intersection of the corresponding column and horizontal

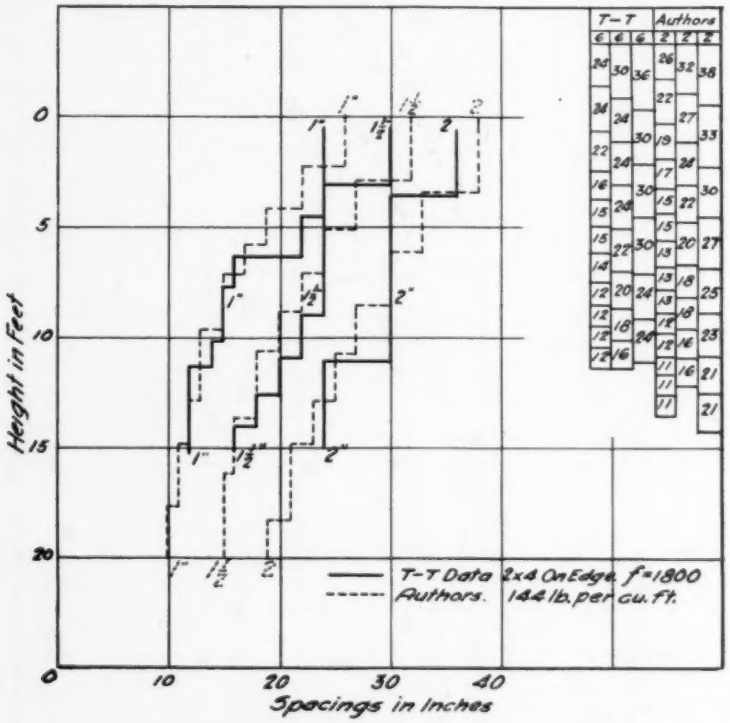



FIG. 8.—COMPARISON OF COLUMN COLLAR SPACINGS.

lines of Table III, respectively, will be the limiting column size for the four sizes of collars indicated, viz: 2 x 4, 2 x 6, 2 x 8, 2 x 10 in. As an example, the maximum size of column for which a 2 x 6 in. collar is suitable for, when used with a $1\frac{1}{2}$ in. sheathing, at a height of 15 ft., is 28 in. for a 72 lb. per cu. ft. but 24 in. for a 144 lb. per cu. ft. If the column in question is of a larger size, either heavier material must be used for the collar, or the collar

spacing determined with a decrease in size of column material, if desired.

## 8. WALLS.

Since a wall form is very similar to a column form, so far as the pressure upon it is concerned, the same spacing as given for

TABLE III.—SPACING OF WALL TIES. USE FIGURES IN ITALIC.

For 72 lb. per cu. ft.

| Size of Studs, in. | Thickness of Sheathing, in. | Depth. Deflection. 1 | 5 | 10 | 15 | 20 | 25 | 30 | 40 | Depth. Strength 1 | 5 | 10 | 15 | 20 | 25 | 30 | 40 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 x 4 | 1 | 50 | 37 | 32 | 30 | 28 | 27 | 26 | 25 | *45* | *30* | *25* | *23* | *21* | *20* | *19* | *18* |
| | 1½ | 46 | 34 | 30 | 28 | 26 | 25 | 24 | 23 | *37* | *25* | *21* | *19* | *17* | *16* | *15* | *14* |
| | 2 | 44 | 32 | 28 | 26 | 25 | 24 | 23 | 22 | *32* | *21* | *18* | *16* | *15* | *14* | *13* | *13* |
| 2 x 6 | 1 | 67 | 50 | 44 | 41 | 38 | 36 | 35 | 34 | *67* | *45* | *38* | *34* | *32* | *30* | *29* | *27* |
| | 1½ | 62 | 46 | 41 | 38 | 36 | 34 | 33 | 31 | *55* | *37* | *31* | *28* | *26* | *25* | *25* | *22* |
| | 2 | 59 | 44 | 38 | 36 | 32 | 32 | 31 | 30 | *48* | *32* | *26* | *24* | *23* | *21* | *20* | *19* |
| 2 x 8 | 1 | 84 | 62 | 54 | 50 | 48 | 46 | 44 | 42 | 90 | *60* | *51* | *46* | *42* | *40* | *38* | *36* |
| | 1½ | 77 | 57 | 50 | 47 | 44 | 42 | 41 | 39 | *73* | *49* | *41* | *37* | *35* | *33* | *31* | *29* |
| | 2 | 73 | 54 | 48 | 44 | 42 | 40 | 39 | 37 | *65* | *43* | *36* | *32* | *30* | *28* | *27* | *25* |
| 2 x 10 | 1 | 99 | 73 | 64 | 59 | 56 | 54 | 52 | 49 | 112 | *75* | *64* | *57* | *53* | *50* | *48* | *45* |
| | 1½ | 92 | 68 | 59 | 55 | 52 | 50 | 48 | 46 | *92* | *61* | *52* | *47* | *43* | *41* | *39* | *36* |
| | 2 | 87 | 64 | 56 | 62 | 49 | 47 | 46 | 43 | *80* | *53* | *45* | *40* | *38* | *36* | *34* | *32* |

For 144 lb. per cu. ft.

| | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 x 4 | 1 | 44 | 32 | 28 | 26 | 25 | 24 | 23 | 22 | *38* | *25* | *21* | *19* | *18* | *17* | *16* | *15* |
| | 1½ | 40 | 30 | 26 | 24 | 23 | 22 | 21 | 20 | *31* | *21* | *17* | *16* | *15* | *14* | *13* | *12* |
| | 2 | 38 | 28 | 25 | 23 | 22 | 21 | 20 | 19 | *27* | *18* | *15* | *14* | *13* | *12* | *11* | *10* |
| 2 x 6 | 1 | 59 | 44 | 38 | 35 | 34 | 32 | 31 | 30 | *57* | *38* | *32* | *29* | *27* | *25* | *24* | *23* |
| | 1½ | 55 | 41 | 36 | 33 | 31 | 30 | 29 | 27 | *46* | *31* | *26* | *24* | *22* | *21* | *20* | *18* |
| | 2 | 52 | 38 | 34 | 31 | 30 | 28 | 27 | 26 | *40* | *27* | *23* | *20* | *19* | *18* | *17* | *16* |
| 2 x 8 | 1 | 74 | 54 | 48 | 44 | 40 | 40 | 39 | 37 | 76 | *51* | *43* | *38* | *36* | *34* | *32* | *30* |
| | 1½ | 68 | 50 | 44 | 41 | 39 | 37 | 36 | 34 | *62* | *41* | *35* | *31* | *29* | *28* | *26* | *24* |
| | 2 | 64 | 47 | 42 | 39 | 37 | 35 | 34 | 32 | *54* | *36* | *30* | *27* | *25* | *24* | *23* | *21* |
| 2 x 10 | 1 | 87 | 64 | 56 | 52 | 49 | 47 | 46 | 44 | 95 | *63* | *53* | *48* | *45* | *42* | *40* | *38* |
| | 1½ | 80 | 60 | 52 | 48 | 46 | 43 | 42 | 40 | *77* | *52* | *44* | *39* | *37* | *35* | *33* | *31* |
| | 2 | 76 | 56 | 50 | 46 | 43 | 42 | 41 | 38 | *67* | *45* | *38* | *34* | *32* | *30* | *29* | *26* |

column collars can be used for the spacing of the studs of a wall form; especially so as said spacing checks up very well with general practice. (See Appendix.—2. Walls.)

In calculating the spacing of the necessary rangers, ties, or otherwise, that hold the forms together, the total load carried

by the studs depends upon the spacing of the studs as well as upon the spacing of the ties, both varying with the height. If $l$ represents the spacing of the ties, and $s$ the spacing of the studs, then the total load $W$ upon a stud between ties will be:

$$W = pls \text{ or } \tfrac{1}{2}\, lsh \text{ for heavy walls}$$
$$\text{and } lsh \text{ for light walls}$$

For deflection a value of $\frac{1}{16}$ of an inch has been adopted. In the Appendix (3. Spacing of Wall Ties) a set of equations has been

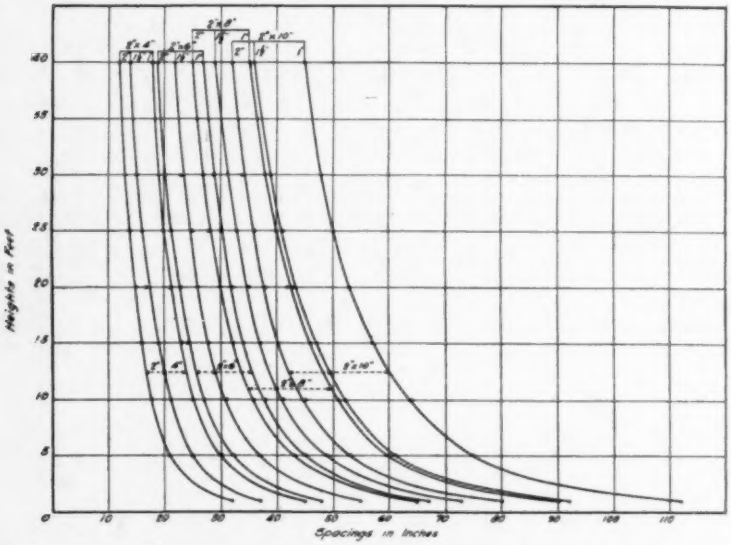



FIG. 9.—SPACING OF WALL TIES FOR STRENGTH. WEIGHT OF WET CONCRETE 72 LB. PER CU. FT. PLOTTED FROM TABLE III.

deducted giving the proper spacing of the ties or otherwise for 2 x 4, 2 x 6, 2 x 8 and 2 x 10-in. studs. The results of solving the wall equations are given in Table III. The same results are shown in forms of curves in Figs. 9 and 10. Care must be taken to see that the ties are strong enough to carry the load brought upon them by these spacings or else trouble will arise.

In the same way that a consecutive spacing table was worked out for collars and studs, a similar table was worked up for spacing

of wall ties as seen in Table IV, though the corresponding curves, similar to Fig. 6 and 7 have not been inserted.

Some concrete experts have stated that a maximum stud spacing for ordinary form structures of:

24 in. for 1-in. sheathing
48 in. " $1\frac{1}{2}$-in. "
60 in. " 2-in. "

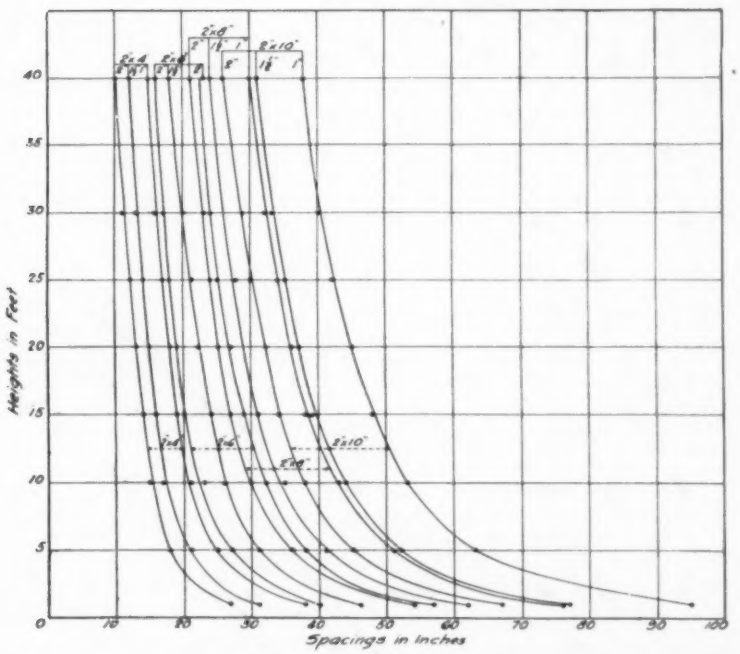



FIG. 10.—SPACING OF WALL TIES FOR STRENGTH. WEIGHT OF WET CONCRETE 144 LB. PER CU. FT. PLOTTED FROM TABLE III.

to be on the side of safety. The author is of the opinion that if a form built of 2-in. sheathing with studs spaced 5 ft. apart was filled at any speed the results would be somewhat disastrous and rather expensive. Such spacings are beyond anything that he has ever dared to attempt in any of his work. As a matter of fact, in a 20-ft. wall a spacing of about 3 ft. was found to be all that 2-in. dressed lumber would stand without signs of springing.

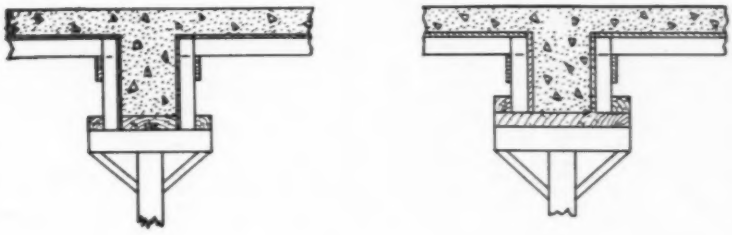
## TABLE IV.—SPACING OF WALL TIES.

| | For 72 lb. per cu. ft. | | | | | | | | | | | | For 144 lb. per cu. ft. | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Studs..... | 2 x 4 | | | 2 x 6 | | | 2 x 8 | | | 2 x 10 | | | 2 x 4 | | | 2 x 6 | | | 2 x 8 | | | 2 x 10 | | |
| Sheathing. | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 |
| From Top of Wall Down, inches. | 35 | 30 | 26 | 48 | 41 | 37 | 60 | 51 | 46 | 72 | 61 | 54 | 30 | 26 | 23 | 41 | 36 | 32 | 52 | 45 | 40 | 62 | 54 | 48 |
| | 29 | 25 | 23 | 41 | 35 | 31 | 52 | 44 | 40 | 62 | 53 | 47 | 25 | 22 | 20 | 36 | 30 | 27 | 45 | 38 | 34 | 53 | 46 | 41 |
| | 27 | 23 | 20 | 37 | 32 | 28 | 48 | 40 | 36 | 56 | 48 | 42 | 23 | 19 | 18 | 33 | 28 | 25 | 41 | 34 | 31 | 49 | 41 | 37 |
| | 25 | 21 | 18 | 35 | 30 | 26 | 44 | 37 | 33 | 52 | 45 | 38 | 22 | 18 | 17 | 31 | 26 | 23 | 38 | 32 | 29 | 46 | 39 | 35 |
| | 23 | 21 | 18 | 32 | 28 | 24 | 40 | 35 | 31 | 48 | 42 | 38 | 21 | 17 | 16 | 29 | 25 | 22 | 36 | 31 | 28 | 44 | 37 | 33 |
| | 23 | 19 | 17 | 32 | 26 | 24 | 40 | 35 | 31 | 48 | 40 | 35 | 20 | 17 | 15 | 28 | 24 | 21 | 36 | 30 | 27 | 42 | 36 | 32 |
| | 21 | 19 | 17 | 30 | 26 | 22 | 37 | 33 | 29 | 45 | 40 | 35 | 19 | 16 | 15 | 27 | 23 | 20 | 34 | 29 | 26 | 40 | 35 | 31 |
| | 21 | 19 | 16 | 30 | 24 | 22 | 37 | 33 | 29 | 45 | 38 | 35 | 19 | 16 | 14 | 27 | 23 | 20 | 34 | 28 | 25 | 40 | 34 | 30 |
| | 21 | 17 | 16 | 30 | 24 | 22 | 37 | 31 | 27 | 45 | 38 | 32 | 18 | 15 | 14 | 26 | 22 | 19 | 32 | 27 | 24 | 38 | 33 | 29 |
| | 20 | 17 | 15 | 28 | 24 | 21 | 35 | 31 | 27 | .. | 36 | 32 | 18 | 15 | 14 | 25 | 21 | 19 | 32 | 27 | 24 | 38 | 32 | 28 |
| | 20 | 17 | 15 | 28 | 24 | 21 | 35 | 29 | 27 | .. | 36 | 32 | 18 | 15 | 13 | 25 | 21 | 18 | 31 | 26 | 23 | .. | 32 | 28 |
| | 20 | 16 | 15 | 28 | 22 | 21 | .. | 29 | 25 | .. | .. | 32 | 17 | 15 | 13 | 24 | 21 | 18 | 31 | 26 | 23 | .. | 31 | 27 |
| | 19 | 16 | 15 | 27 | 22 | 20 | .. | 29 | 25 | .. | .. | .. | 17 | 14 | 13 | 24 | 20 | 18 | 30 | 25 | 22 | .. | .. | 27 |
| | 19 | 16 | 14 | 27 | 22 | 20 | .. | .. | 25 | .. | .. | .. | 17 | 14 | 13 | 23 | 20 | 17 | .. | 25 | 22 | .. | .. | 27 |
| | 19 | 15 | 14 | 27 | 22 | 20 | .. | .. | 25 | .. | .. | .. | 17 | 14 | 13 | 23 | 20 | 17 | .. | 24 | 22 | .. | .. | 26 |
| | 19 | 15 | 14 | .. | 22 | 19 | .. | .. | .. | .. | .. | .. | 16 | 14 | 12 | 23 | 19 | 17 | .. | 24 | 21 | .. | .. | .. |
| | 18 | 15 | 14 | .. | 22 | 19 | .. | .. | .. | .. | .. | .. | 16 | 14 | 12 | 23 | 19 | 17 | .. | .. | 21 | .. | .. | .. |
| | 18 | 15 | 13 | .. | 22 | 19 | .. | .. | .. | .. | .. | .. | 16 | 13 | 12 | .. | 19 | 17 | .. | .. | 21 | .. | .. | .. |
| | 18 | 15 | 13 | .. | .. | 19 | .. | .. | .. | .. | .. | .. | 16 | 13 | 12 | .. | 18 | 16 | .. | .. | .. | .. | .. | .. |
| | 18 | 14 | 13 | .. | .. | 19 | .. | .. | .. | .. | .. | .. | 16 | 13 | 12 | .. | 18 | 16 | .. | .. | .. | .. | .. | .. |
| | 18 | 14 | 13 | .. | .. | 19 | .. | .. | .. | .. | .. | .. | 15 | 13 | 12 | .. | 18 | 16 | .. | .. | .. | .. | .. | .. |
| | 18 | 14 | 13 | .. | .. | .. | .. | .. | .. | .. | .. | .. | 15 | 13 | 11 | .. | .. | 16 | .. | .. | .. | .. | .. | .. |
| | .. | 14 | 13 | .. | .. | .. | .. | .. | .. | .. | .. | .. | 15 | 13 | 11 | .. | .. | 16 | .. | .. | .. | .. | .. | .. |
| | .. | 14 | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | 15 | 12 | 11 | .. | .. | 16 | .. | .. | .. | .. | .. | .. |
| | .. | 14 | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | 15 | 12 | 11 | .. | .. | 16 | .. | .. | .. | .. | .. | .. |
| | .. | 14 | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 12 | 11 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | 14 | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 12 | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 12 | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 12 | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 12 | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 12 | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | 10 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| Balance... | 11 | 3 | 7 | 0 | 12 | 7 | 15 | 23 | 24 | 7 | 3 | 28 | 9 | 1 | 1 | 13 | 10 | 10 | 8 | 9 | 17 | 28 | 30 | 18 |
| Total..... | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 | 480 |

## 9. FLOOR BEAMS.

In floor beams the lower plank becomes the principal member to study. Beam forms can be built in two ways: (*a*) with the bottom board between the side forms; and (*b*) with the bottom board extended outside the side forms.

In the first case the width of the beam makes no difference in the amount of deflection or the extreme outside fiber stress of the bottom board, as the load upon each inch of its width is dependent only upon the depth of the beam. In the second case the deflection and stress depend upon both the width of the beam and its depth. The wider the beam the less in proportional width is the bottom



(*a*) (*b*)

board. A maximum deflection of $\frac{1}{8}$ of an inch has been adopted for the allowable deflection of the bottom member of a beam form.

The total loading upon the lower member of a beam form is the total weight of the wet concrete in the form plus an assumed live load. By adopting a weight of 144 lb. per cu. ft. for the wet concrete and a live load of 48 lb. to the sq. ft. the total load upon the bottom member of the beam form per foot of length becomes,

$$w \text{ (load per foot of length)} = WD + \frac{48}{12} W \text{ or } W\ (D+4),$$

where $W$ equals the width and $D$ the depth of the beam in inches.

Table V gives values of $w$ (weight of concrete per lineal foot) for various size beams, from 6 to 60 in. in depth, 6 to 30 in. in width, in terms of dead, live and total loads.

If $L$ equals the distance between supports in inches, then the

total load ($Wt$) upon the lower member of the beam form between supports will be,

$$Wt \text{ (total load upon form)} = \frac{L}{12} w \text{ or } \frac{LW\ (D+4)}{12}$$

The substitution of this expression for $Wt$ in equations (A) and (B) results in two simple expressions for determining the

TABLE V.—LOAD ON BEAMS. BASED UPON 1 CU. FT. CONCRETE = 144 LB WIDTH AND DEPTH OF BEAM IN INCHES.

Weight ($w$) of Concrete per Linear Foot of Beam, lb.

| | Width, in. | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 21 | 24 | 27 | 30 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Dead Load. | Beam Depth, inches. 6 | 36 | 48 | 60 | 72 | 84 | 96 | 108 | 126 | 144 | 162 | 180 |
| | 12 | 72 | 96 | 120 | 144 | 168 | 192 | 216 | 252 | 288 | 324 | 360 |
| | 18 | 108 | 144 | 180 | 216 | 252 | 288 | 324 | 378 | 432 | 486 | 540 |
| | 24 | 144 | 192 | 240 | 288 | 336 | 384 | 432 | 504 | 576 | 648 | 720 |
| | 30 | 180 | 240 | 300 | 360 | 420 | 480 | 540 | 630 | 720 | 810 | 900 |
| | 36 | 216 | 288 | 360 | 432 | 504 | 576 | 648 | 756 | 864 | 972 | 1080 |
| | 42 | 252 | 376 | 420 | 504 | 588 | 672 | 756 | 882 | 1008 | 1134 | 1260 |
| | 48 | 288 | 384 | 480 | 576 | 672 | 668 | 864 | 1008 | 1152 | 1296 | 1440 |
| | 54 | 324 | 432 | 540 | 648 | 756 | 854 | 972 | 1134 | 1296 | 1458 | 1620 |
| | 60 | 360 | 480 | 600 | 720 | 840 | 960 | 1080 | 1260 | 1440 | 1620 | 1800 |
| Live Load at 48 lb. per sq. ft. | | 24 | 32 | 40 | 48 | 56 | 64 | 72 | 84 | 96 | 108 | 120 |
| Dead and Live Loads. | Beam Depth, inches. 6 | 60 | 80 | 100 | 120 | 140 | 160 | 180 | 210 | 240 | 270 | 300 |
| | 12 | 96 | 128 | 160 | 192 | 224 | 256 | 288 | 336 | 384 | 432 | 480 |
| | 18 | 132 | 176 | 220 | 264 | 308 | 352 | 396 | 462 | 528 | 594 | 660 |
| | 24 | 168 | 224 | 280 | 336 | 392 | 448 | 504 | 588 | 672 | 756 | 840 |
| | 30 | 204 | 272 | 340 | 408 | 476 | 544 | 612 | 714 | 816 | 918 | 920 |
| | 36 | 240 | 320 | 400 | 480 | 560 | 640 | 720 | 840 | 960 | 1080 | 1200 |
| | 42 | 276 | 408 | 460 | 552 | 664 | 736 | 828 | 866 | 1104 | 1242 | 1380 |
| | 48 | 312 | 416 | 520 | 624 | 728 | 832 | 936 | 1092 | 1248 | 1404 | 1560 |
| | 54 | 348 | 564 | 580 | 696 | 812 | 918 | 1044 | 1218 | 1392 | 1566 | 1740 |
| | 60 | 384 | 640 | 640 | 768 | 896 | 1024 | 1152 | 1344 | 1536 | 1728 | 1920 |

proper spacing of struts under a beam form built with a 2-in. bottom member, for both types of construction based upon a dead load of 144 lb. per cu. ft. and a live load of 48 lb. per sq. ft., as found in Appendix (4. Beams). Table VI gives the results of solving the beam equations for both types of construction; the strut spacing for the inside type of form being closer than for the outside type. In the outside type where the ratio of the form width to its bottom board changes the strut spacing is the same for several depths in a great many cases.

The author recommends the spacing for the inside type as given in Table VI and as plotted in Fig. 11 covering all cases, for

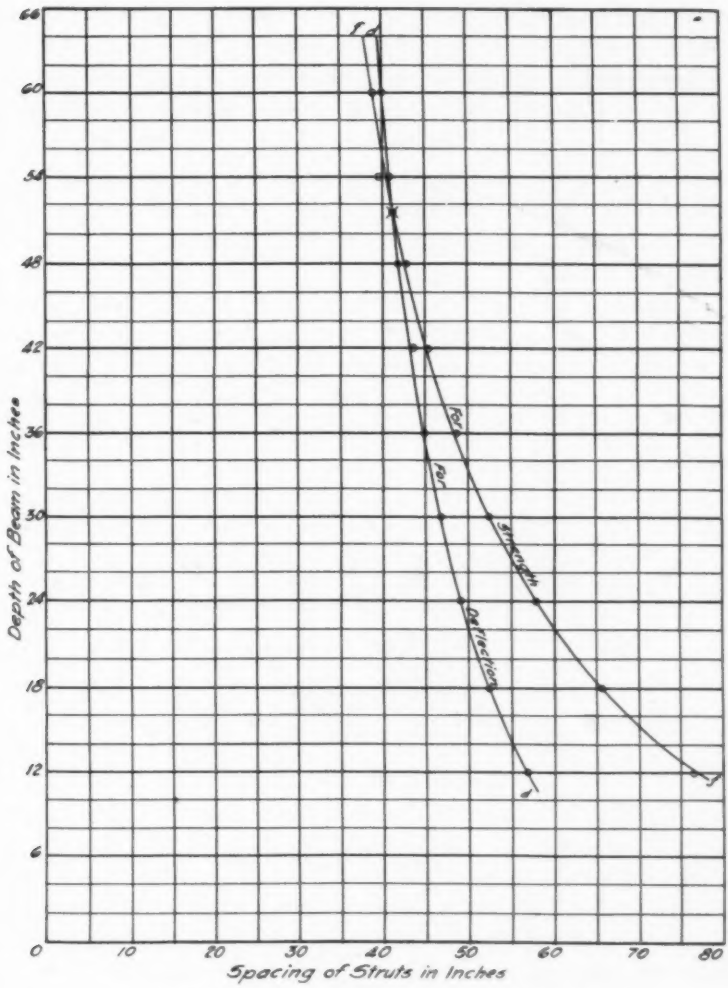
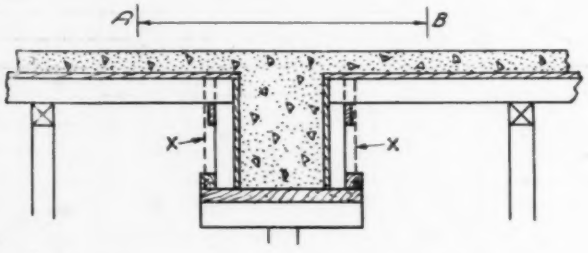



FIG. 11.—SPACING OF STRUTS UNDER BEAMS.

this reason. The weight of the floor slab between the floor beam and the first line of girders supporting the floor joists is carried half by the beam and half by the floor girder. This is only true

of the beams which carry the ends of the floor joists, not of the beams that are parallel to the floor joists. In the first instance, with a 24-in. spacing to the floor form joist and a 48-in. spacing to the beam struts, it brings one of the floor joists over the center of a beam span, as it were, thus putting an additional load upon the bottom between its points of support, that is, the load carried by one floor joist, hence increasing its deflection or fiber stress. Possibly the side form of the beam can be viewed in the light of a web girder supported at the struts; but it has not been so considered. The floor joists over the struts bring no bending effect upon the bottom board, but do increase the load upon the struts under the beam.

In the second case the additional load brought upon the beam form is that of one-half of a floor joist spacing uniformly distributed over the sides of the beam form, being of a less magnitude than in the first case. Thus the provision adopted to carry the additional load of case (*a*) will more than suffice in case (*b*). The following sketch illustrates more plainly what is meant:




On the supposition that the weight of the concrete slab between *A*—*B* is carried by the beam form, the dotted lines (x) show how much would have to be added to and made a part of the beam in order to have a beam of a size containing the amount of concrete which the supports to the original beam carry. The additional concrete makes the beam as wide as the bottom board. A case of an inside type of form. It very plainly shows why an inside type of form spacing (as given in Table VI) for spacing the struts under all beams has been recommended.

Though the spacing of the struts may be proper from a deflection or strength point of view, the load upon them may be in excess

of the allowable loads as noted in a preceding table. The following table has been made out for five different widths of beams whose depth is twice their width. The weight per lineal foot is that given in Table V; the strut spacing as given in Table VI; the total loads per strut being the result of multiplying column 3 by 4.

A comparison of the total loads per strut of this table with the allowable loads as previously noted will show them to be well inside the limits of safety, that is, for a 4 x 4 or larger size strut.

TABLE VI.—BEAMS. USE FIGURES IN ITALIC.

| | | Spacing of Struts, inches. | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Type. | | Inside. | | Outside. | | | | | | | | | | | |
| *d* or *f* | | Defl | St. | B | Deflection. | | | | | Strength. | | | | | |
| Width. | | All. | | | 6 | 12 | 18 | 24 | 30 | 6 | 12 | 18 | 24 | 30 | |
| Depth, inches. | 12 | *57* | 77 | 10 | *73* | *66* | *63* | *62* | *61* | 126 | 105 | 97 | 92 | 89 | |
| | 18 | *53* | 66 | 10 | *67* | *61* | *59* | *57* | *56* | 108 | 89 | 82 | 83 | 76 | |
| | 24 | *49* | 59 | 10 | *63* | *57* | *55* | *54* | *53* | 92 | 79 | 73 | 70 | 68 | |
| | 30 | *47* | 53 | 10 | *61* | *45* | *53* | *51* | *50* | 84 | 72 | 66 | 63 | 61 | |
| | 36 | *45* | 49 | 14 | *61* | *55* | *52* | *51* | *50* | 89 | 72 | 65 | 62 | 59 | |
| | 42 | *44* | 46 | 18 | *61* | *55* | *52* | *50* | *49* | 91 | 72 | 65 | 60 | 58 | |
| | 48 | *42* | 43 | 18 | *60* | *53* | *50* | *49* | *48* | 86 | 68 | 61 | 57 | 54 | |
| | 54 | 41 | *41* | 22 | *60* | *53* | *50* | *49* | *47* | 88 | 68 | 61 | 56 | 53 | |
| | 60 | 40 | *39* | 22 | *59* | *52* | *49* | *48* | *46* | 84 | 65 | 58 | 54 | 50 | |

B = Width of bottom board of form in excess of beam width based upon a liquid weight of 144 lb. per cu. ft. and a live load of 48 lb. per sq. ft.

RESULTING LOADS UPON STRUTS.

| Depth of Beam, | Width of Beam, | Weight per Lineal Foot, | Spacing of Struts, | Total Loads per Strut, |
|---|---|---|---|---|
| 12 | 6 | 96 | 57 | 450 |
| 24 | 12 | 336 | 49 | 1,372 |
| 36 | 18 | 720 | 45 | 2,700 |
| 48 | 24 | 1,248 | 42 | 4,368 |
| 60 | 30 | 1,920 | 39 | 6,240 |

As already mentioned, part of the floor slab load is carried by the beam struts. Just how much depends upon the spacing of the girders to the floor slab form. As shown in the above sketch, the total amount of concrete carried by the beam strut is about twice that due to the beam itself, which would thus double the strut loads as tabulated. Even then the resulting loads would be less than given in the table of allowable strut loads, and so are on the side of safety.

The side members of a beam are nothing more nor less than a wall form and can be so considered; the necessary data for determining the stud spacing being found in Tables I and II.

## 10. FLOOR SLABS.

Floor slab forms as generally built consist of four different items, viz: the sheathing, the joists, the girders, and the struts. The point of the greatest deflection from a true flat surface is theoretically centrally located as respects the panels into which the form is divided by the supporting struts. If a maximum deflection of $\frac{1}{32}$ in. is allowed for the flooring between the joists, a deflection of $\frac{1}{32}$ in. for the joists between the girders and the same amount of deflection for the girders between their supports, the total theoretical deflection for the above centrally located point will be $\frac{3}{32}$ in. A deflection of $\frac{1}{32}$ in. has been taken for all members of the floor slab form.

The load upon the slab forms is the total weight of the wet concrete plus the dead load as in the case of a beam. Factors of 144 and 72 lb. per sq. ft. have been taken for the dead and live loads to simplify the deductions of the equations, and hence the calculations. If $W$ represent the total load per sq. ft. in lb. (including both live and dead loads) and $T$ the thickness of the concrete slab in inches, then

$$W \text{ (total load per sq. ft.)} = \frac{144}{12} T + 72 \text{ or } 12\,(T+6)$$

Substituting this expression in equations (A) and (B), using the same value for $E$ and $f$ as already given, full sets of equations have been deducted covering: 1, spacing of floor joists; 2, spacing of girders; and 3, spacing of struts under the girders for a floor slab form. (See Appendix.—5. Floor Slab; 6. Spacing of Girders; and 7. Spacing of Struts.)

In deducting the floor joist equations the flooring has been considered as a beam 12 in. wide and 1 in. thick, carrying a uniformly distributed total load of $W$ lb., varying according to the thickness of the slab, 1-in. flooring alone being considered.

In the girder equation the joists have been considered as beams carrying a total uniformly distributed load of $W$ lb. depend-

ing upon their spacing and thickness of slab. The sizes of floor joists taken are 2 x 4, 2 x 6, 2 x 8 and 2 x 10 inches.

In the strut equation the girders are likewise considered as a beam carrying a total uniformly distributed load of $W$ lb., depending upon their spacing and thickness of floor slab. A size of 4 x 6 in. has been taken for the girder.

The results of solving the floor slab equations for every 2 in. of thickness from 2 in. up to 12 in. for both deflection and strength are recorded in Table VII; all based upon thickness of floor slab

TABLE VII.—FLOOR SLABS. USE FIGURES IN ITALIC.

Spacing of Joists—Girders and Struts for Floor Slabs.

| Items. | | | For Deflection. | | | | | | For Strength. | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Thickness of Slab, inches | | | 2 | 4 | 6 | 8 | 10 | 12 | 2 | 4 | 6 | 8 | 10 | 12 |
| Loads, lb. per sq. ft. | Dead | | *24* | *48* | *72* | *96* | *120* | *144* | *24* | *48* | *72* | *96* | *120* | *144* |
| | Live | | *72* | *72* | *72* | *72* | *72* | *72* | *72* | *72* | *72* | *72* | *72* | *72* |
| | Total | | *96* | *120* | *144* | *168* | *192* | *216* | *96* | *120* | *144* | *168* | *192* | *216* |
| Spacing of Joists | | 1-in. Flooring | *28* | *26½* | *25½* | *24½* | *24* | *23* | 55 | 49 | 45 | 41½ | 39 | 36½ |
| Spacing of Girders. | 1-in. Flooring throughout | 2 x 4 | *41½* | 40 | 38 | 37 | 36 | 35½ | 42 | *39½* | *37½* | *36* | *35* | *34½* |
| | | 2 x 6 | *56* | *54* | *52* | *50½* | *49* | *48* | 62½ | 59 | 56½ | 54½ | 52½ | 51½ |
| | | 2 x 8 | *70* | *67* | *64½* | *62½* | *61* | *60* | 83½ | 79 | 75½ | 72½ | 71 | 69 |
| | | 2 x 10 | *82* | *79* | *76* | *74* | *73* | *70½* | 104 | 99 | 94 | 91 | 88 | 86 |
| Spacing of Struts for 4-x 6-in. Girders. | 1-in. Flooring throughout | 2 x 4 | *60½* | *58* | *56* | *54* | *53* | *51½* | 103 | 94 | 89 | 84 | 80 | 76 |
| | | 2 x 6 | *86* | *84* | *82* | *80* | *49* | *48* | 84 | 77 | 72 | 68 | 65 | 62 |
| | | 2 x 8 | *66* | *51* | *49* | *47½* | *46* | *45* | 73 | 67 | 63 | 59 | 56 | 54 |
| | | 2 x 10 | *51* | *49* | *47* | *45½* | *44½* | *43* | 65 | 60 | 56 | 53 | 50 | 48 |

and figured for a total load as noted at the top of the table. The same data is shown in plotted form in Fig. 12.

For a given thickness of floor slab the thicker the flooring the greater the spacing of the joists, but the closer the spacing of the girders carrying the joists due to the fact that the wide spaced joists will be carrying more load than the narrowly spaced joists. Consequently for any given thickness of a floor slab the thickness of the flooring becomes the starting point in determining the spacing of the other members of the floor slab form structure.

In order to reduce the above results to a simple table suitable for office use as well as for the job force, Table VIII has been made up. It is felt that this table will furnish the engineer as well as

TABLE VIII.—SPACING OF FORM MEMBERS.

| Height Ft. | SPACING OF WALL TIES | | | | | | | | | | | | | | | | | | | | | | | | Height Ft. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | FOR 72 LB. PER CU. FT. | | | | | | | | | | | | FOR 144 LB. PER CU. FT. | | | | | | | | | | | | |
| | 2 × 4 | | | 2 × 6 | | | 2 × 8 | | | 2 × 10 | | | 2 × 4 | | | 2 × 6 | | | 2 × 8 | | | 2 × 10 | | | |
| | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | 1 | 1½ | 2 | |
| 1 | | | | | | | | | | | | | | | | | | | | | | | | | 1 |
| 2 | 35 | 30 | 26 | 48 | 41 | 37 | 60 | 51 | 46 | 72 | 61 | 54 | 30 | 26 | 23 | 14 | 36 | 32 | 52 | 45 | 40 | 62 | 54 | 48 | 2 |
| 3 | | | | | | | | | | | | | | | 20 | | | | | | | | | | 3 |
| 4 | | 25 | 23 | | | | | | | | | | 25 | 22 | | | | 27 | | | | | | | 4 |
| 5 | 29 | | | | | 31 | | | | | | | | 19 | 18 | 36 | 30 | | | | | | | | 5 |
| 6 | | 23 | 20 | 47 | 35 | | | | 40 | | | | 23 | | | | | 24 | | 38 | 34 | | | | 6 |
| 7 | 27 | | 18 | | | 28 | 52 | 44 | | | | 47 | | 18 | 17 | | 29 | | 46 | | | | 46 | 41 | 7 |
| 8 | | 21 | | | 32 | | | | 36 | | 53 | | 22 | 17 | 16 | 33 | | 25 | | 34 | 31 | 55 | | | 8 |
| 9 | 25 | | 18 | 37 | | 26 | 62 | 40 | | | | 42 | | | 15 | | 26 | | 41 | | | | 37 | | 9 |
| 10 | | 21 | 17 | | 30 | | | | 33 | | 48 | | 21 | 17 | 15 | 31 | | 23 | | 32 | 29 | | 47 | | 10 |
| 11 | 23 | 19 | 17 | 35 | | 24 | 48 | 37 | | | | 38 | 20 | 16 | 14 | | 25 | 21 | | | | 49 | | 35 | 11 |
| 12 | | | | | 28 | | | | 31 | 56 | 45 | | | 16 | 14 | 29 | 24 | 20 | 38 | 31 | 27 | | 39 | | 12 |
| 13 | 23 | 19 | 16 | | | 24 | | 37 | | | | 38 | 19 | 15 | 14 | | | | | | | | | 33 | 13 |
| 14 | | 19 | 16 | 32 | 26 | 22 | 44 | 35 | 31 | | 45 | | 19 | 15 | 13 | 28 | 23 | 20 | 36 | 30 | 26 | 46 | 37 | | 14 |
| 15 | 21 | 17 | 15 | | | | | | | 52 | 42 | 38 | 18 | 15 | 13 | 27 | 22 | 19 | | | | | | 38 | 15 |
| 16 | 21 | 17 | 15 | 32 | 26 | 22 | 40 | 35 | 31 | | | | 18 | 15 | 13 | | | | 35 | 29 | 26 | 44 | 36 | | 16 |
| 17 | 21 | 17 | 15 | | | | | | | | | 35 | 18 | 14 | 12 | 27 | 22 | 19 | | | | | | 34 | 17 |
| 18 | | | | 30 | 24 | 22 | | 33 | 29 | 48 | 40 | | 17 | 14 | 12 | 26 | 21 | 18 | 34 | 28 | 24 | 42 | 35 | 30 | 18 |
| 19 | 20 | 16 | 14 | | | | 40 | | | | | 33 | 17 | 14 | 12 | | | | | | | | | | 19 |
| 20 | 20 | 16 | 14 | 30 | 24 | 21 | | 33 | 29 | | 40 | | 17 | 14 | 12 | 26 | 21 | 18 | 34 | 28 | 24 | 42 | 35 | 30 | 20 |
| 21 | | | | | | | | | | 48 | | 33 | 17 | 14 | 12 | | | | | | | | | | 21 |
| 22 | 20 | 16 | 14 | 30 | 24 | 21 | 40 | 33 | 29 | | 40 | | 17 | 14 | 12 | 26 | 21 | 18 | 34 | 28 | 24 | 42 | 35 | 30 | 22 |
| 23 | | | | | | | | | | 48 | | 33 | 17 | 14 | 12 | | | | | | | | | | 23 |
| 24 | 20 | 16 | 14 | 30 | 24 | 21 | 40 | 33 | 29 | | 40 | | 17 | 13 | 12 | 26 | 21 | 18 | 34 | 28 | 24 | 42 | 35 | 30 | 24 |
| 25 | | | | | | | | | | 48 | | 33 | | | | | | | | | | | | | 25 |
| 26 | | 15 | 13 | | 24 | 21 | | 33 | 29 | | 40 | | 17 | 13 | 12 | 26 | 21 | 18 | 34 | 28 | 24 | 42 | 35 | 30 | 26 |
| | | | | | | | | | | 48 | | 33 | | | | | | | | | | | | | |

TABLE VIII.—SPACING OF FORM MEMBERS. (*Continued.*)

BEAM STRUTS.

| Depth, inches. | Inside Type. | Outside Type. | | | | |
|---|---|---|---|---|---|---|
| | | Width, inches. | | | | |
| | All. | 6 | 12 | 18 | 24 | 30 |
| 12 | 57 | 73 | 66 | 63 | 62 | 61 |
| 18 | 53 | 67 | 61 | 59 | 57 | 56 |
| 24 | 49 | 63 | 57 | 55 | 54 | 53 |
| 30 | 47 | 61 | 55 | 53 | 51 | 50 |
| 36 | 45 | 61 | 55 | 52 | 51 | 50 |
| 42 | 44 | 61 | 55 | 52 | 50 | 49 |
| 48 | 42 | 60 | 53 | 50 | 49 | 48 |
| 54 | 41 | 60 | 53 | 50 | 49 | 47 |
| 60 | 39 | 59 | 52 | 49 | 48 | 46 |

FLOOR SLABS.

| Thickness of Slab, inches. | | | 2 | 4 | 6 | 8 | 10 | 12 |
|---|---|---|---|---|---|---|---|---|
| Spacing of Joists. | | 1-in. Flooring. | 28 | 26½ | 25½ | 24½ | 24 | 23 |
| Spacing of Girders. | 1-in. Flooring Throughout. | Size of Joists. | | | | | | |
| | | 2 x 4 | 41½ | 39½ | 37½ | 36 | 35 | 34½ |
| | | 2 x 6 | 56 | 54 | 52 | 50½ | 49 | 48 |
| | | 2 x 8 | 70 | 67 | 64½ | 62½ | 61 | 60 |
| | | 2 x 10 | 82 | 79 | 76 | 74 | 72 | 70½ |
| Spacing of Struts for 4 x 6 Girders. | | 2 x 4 | 60½ | 58 | 56 | 54 | 53 | 51½ |
| | | 2 x 6 | 56 | 54 | 52 | 50 | 49 | 48 |
| | | 2 x 8 | 53 | 51 | 49 | 47½ | 46 | 45 |
| | | 2 x 10 | 51 | 49 | 47 | 45½ | 44½ | 43 |

the superintendent and his foremen a clear, concise, statement covering the spacing of form members.

The author feels that the above discussion thoroughly covers the subject and fully illustrates the fundamental principles of

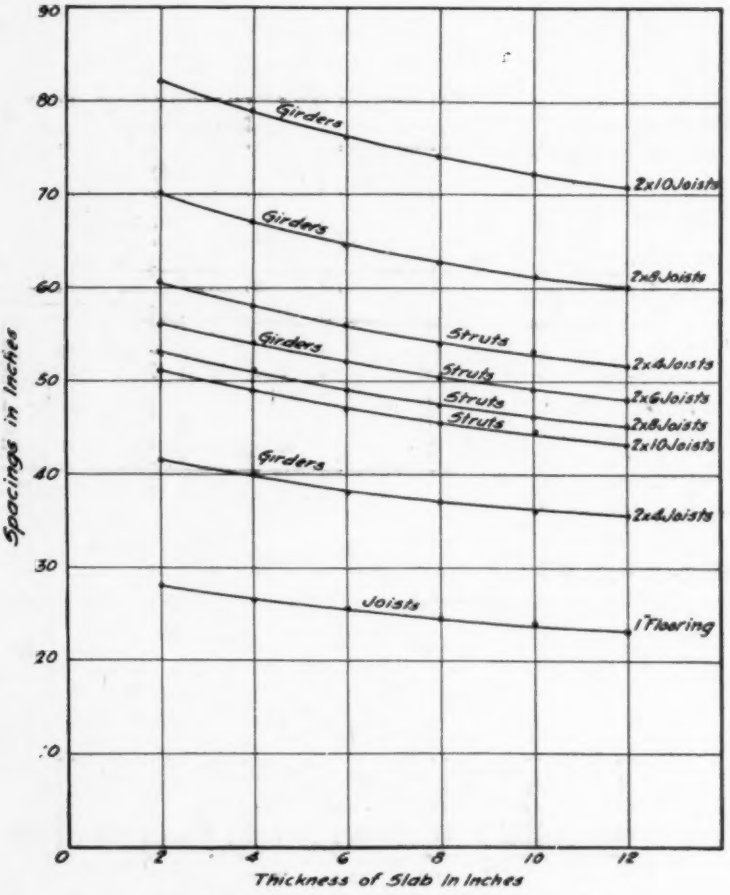



FIG. 12.—FLOOR SLABS. DEFLECTION ONLY.

calculating the strength of a form structure. While a more explanatory discussion of each table and figure might bring out the special features of each, it is considered that a little personal study will accomplish more than many words of the author.

## 11. LIMITS OF STRENGTH, DEFLECTION AND PRESSURE ASSUMPTIONS.

The deflections that have been assumed in making the calculations, viz: $\frac{1}{32}$, $\frac{1}{16}$ and $\frac{1}{8}$ of an inch, do not have the same ratio to the length of the span to which they apply. For instance, a $\frac{1}{8}$ of an inch deflection in the bottom member of a beam form with struts spaced 5 ft. apart will be a much less deflection relative to a 5-ft. spacing than if the same deflection existed between supports 2 ft. apart. In fact the stresses which might be produced in the outer fibers of a beam having a $\frac{1}{8}$ of an inch deflection with a still shorter span, might be such as to cause the failure of the form member in question. Thus the spacing of form members, as figured for deflection, may make the outside fiber stress excessive. On the other hand, spacing them as figured for outside fiber stress may cause excessive deflection, especially if such spacing results in a long span. On the assumption that the smaller of the two spacings must be adopted, for a fixed deflection and for a fixed fiber stress the spacing of the members of a form structure will be, for certain loads, as determined by deflection calculations; for other loads, spacings as figured for allowable stress must be used. The final result is that under certain conditions the assumed deflection and the assumed fiber stress will give the same spacings. This neutral point, as it has been termed, is simply the intersection of the deflection and stress spacing curves, as plainly illustrated in Fig. 3, 5 and 11, and can be determined in two ways:

1. By making a complete calculation for all conditions for both deflection and stress, and determine the neutral point by inspection;

2. By raising the general equation ($G$) to the same power, equating and solving for the unknown quantity.

Both methods have been used in order to illustrate how great a difference there is in the spacings as figured for deflection and for stress, also to obtain an exact mathematical determination of the neutral point. In case of a beam the neutral point works out at 51.4 in. (See Appendix.—8. Beams.) That is when the beam is 51.4 in. deep the spacings of the struts for an inside type (with a 2-in. bottom board) as determined for the assumed deflection and for the assumed outside fiber stress, are the same. For a depth

above 51.4 in. the strut spacings as determined for outside fiber stress should be used; for a depth below 51.4 in. as determined for deflection. In case of a column the neutral point depends upon the thickness of the sheathing as well as upon the depth of the concrete in the column. For a 1-in. sheathing the neutral point is at a height of 9.23; for 2-in. sheathing at a height of 18.46. (See Appendix.—9. Columns.) That is, when the height of the concrete carried by a column collar is approximately 9 ft. for 1-in. sheathing and 18½ ft. for 2-in. sheathing, the spacing of the collars as determined for deflection and stress is the same; all under the assumption that a cu. ft. of wet concrete weighs 72 lb.

Though the calculations for collar spacings and wall studs are based upon two different assumptions, there is a third factor that enters into the case, viz: the pounding action of the dropped concrete and its arch effet. It is possible that the pressure of the concrete upon the forms through such means might be so far in excess of what has been allowed in the calculations as to give poor results to the finished concrete, the collar spacings as recorded in the tables being excessive. Such cases illustrate one of the uncertainties of form work, which are best avoided by using a conservative spacing of the form members and pouring the concrete accordingly.

Form work cannot be figured down to the very last item as it is a type of construction in which it is impossible to do so. To come as near the truth as possible and keep within bounds of safety is all that can be expected.

## 12. CONCLUSION.

In working out these tables and curves the ordinary cases met with in concrete construction have been covered, with the way pointed to a solution of other cases of a similar nature. In heavy arch bridge work or other structures carrying a heavy load, special study of each condition is necessary in order to obtain the best results and to be sure that proper strength has been provided in the temporary wooden structure,—cases that are of such a special nature that it is impossible to go into them in this discussion.

## Appendix.

### 1. Columns. Spacing of Collars.

$d$ = Deflection  Assumed to be $\frac{1}{16}$ in.
$l$ = Spacing of collars—inches.
$b$ = Width of assumed beam = 1 in.  $I = \frac{1}{12} bH^3$
$t$ = Thickness of sheathing = $1\frac{1}{2}$ −2− in.  $f$ = 1200 lb.
$h$ = Height of column above collar—feet.  $E$ = 1,300,000 lb.
$p$ = Pressure, lb. per sq. in.
$P$ = Pressure weight of cu. ft. wet concrete.
$w$ = Total pressure on strip of sheathing 1 in. wide 2 in. long.

| $P$ | $p$ | $W = pl$ |
|---|---|---|
| 72 lb. | $\frac{1}{2}h$ | $\frac{1}{2}lh$ |
| 144 lb. | $h$ | $lh$ |

| For Deflection. | For Strength. |
|---|---|
| $d = \frac{W^3}{1.8\ EI}$ | $Wl = \frac{5}{3}\ bl^2 f$ (1) |

Sub-stituting in Equation (1).

| $P$ | | 72 lb. | 144 lb. | 72 lb. | 144 lb. |
|---|---|---|---|---|---|
| $l^4$ or $l^2$ | | $\frac{5{,}200{,}000l^3}{3h}$ | $\frac{2{,}600{,}000l^3}{3h}$ | $\frac{4000l^2}{h}$ | $\frac{2000l^2}{h}$ (G) |
| Thickness of Sheathing, inches. | 1 | $\frac{5{,}200{,}000}{3h}$ | $\frac{2{,}600{,}000}{3h}$ | $\frac{4000}{h}$ | $\frac{2000}{h}$ |
| | $1\frac{1}{2}$ | $\frac{5{,}850{,}000}{h}$ | $\frac{2{,}925{,}000}{h}$ | $\frac{9000}{h}$ | $\frac{4500}{h}$ |
| | 2 | $\frac{41{,}600{,}000}{3h}$ | $\frac{20{,}800{,}000}{3h}$ | $\frac{16{,}000}{h}$ | $\frac{8000}{h}$ |

* For deflection = $l^4$; For Strength = $l^2$.

## 2. WALLS. SPACING OF STUDS.

$d$ = Deflection. Assumed to be $\frac{1}{16}$ in.
$l$ = Spacing of studs—inches.
$b$ = Width of assumed beam—1 in.
$t$ = Thickness of lumber 1 = in. $1\frac{1}{2}$ = in. 2 = in.
$h$ = Depth of concrete on Stud—feet.
$p$ = Pressure per sq. in. = $\frac{1}{2}h$, or $h$ according to weight of cu. ft. concrete.
$W$ = Total pressure on beam 1 in. wide, l in. long.
$\therefore W = p \times l$ or $\frac{1}{2} l\, h$, for 72 lb. per cu. ft.
$= l\, h$, for 144 lb. per cu. ft.

| For Deflection. | For Strength. | |
|---|---|---|
| $d = \frac{Wl^3}{128\,EI}$ | $Wl = \frac{5}{3}\, bt^2 f$ | (1) |

SUBSTITUTING IN EQUATION (I).

| | | |
|---|---|---|
| $\therefore l^4 = \frac{5{,}200{,}000}{3h}\, t^3$ | $\therefore l^3 = \frac{4{,}000}{h}\, t^3$ | (G) |

Consequently spacing of wall studs equals spacing of column collars for either case.

### 3. SPACING OF WALL TIES.

$d$ = Deflection. Assumed to be $\frac{1}{16}$ in.
$l$ = Spacing of ties—inches.
$s$ = Spacing of studs—inches.
$b$ = Width of studs—2 in.
$H$ = Depth of studs.
$h$ = Height of concrete on studs.
$p$ = Pressure per sq. in. $\frac{1}{2}\ h$ or $h$ according to weight of concrete per cu. ft.
$W$ = Total load on studs.

$\therefore W = p \times s \times l$ or $\frac{1}{2}\ s \times l \times h$ for 72 lb. per cu. ft. (A)
but $s \times l \times h$ for 144 lb. per cu. ft. (B)

SUBSTITUTING (A) IN EQUATION 1.

| | | |
|---|---|---|
| $\therefore l^4 = \frac{10{,}400{,}000 H^3}{3sh}$ | $\therefore l^2 = \frac{8000 H^3}{sh}$ (G) | |
| | But $s = \sqrt{\frac{4000\ l^2}{h}}$ | |
| | $\therefore l^2 = \frac{400\ H^2}{l\sqrt{10h}}$ | |
| $H = 4$ in. $l^4 = \frac{665{,}600{,}000.}{3\ sh}$ | $H = 4$ in. $l^2 = \frac{128{,}000}{sh}$ | or $\frac{6{,}400}{l\sqrt{10h}}$ |
| $H = 6$ in. $1^4 = \frac{2{,}246{,}400{,}000.}{3sh}$ | $H = 6$ in. $l^2 = \frac{288{,}000}{sh}$ | or $\frac{14{,}400}{l\sqrt{10h}}$ |
| $H = 8$ in. $l^4 = \frac{5{,}324{,}800{,}000.}{3sh}$ | $H = 8$ in. $l^2 = \frac{512{,}000}{sh}$ | or $\frac{25{,}600}{l\sqrt{10h}}$ |
| $H = 10$ in. $1^4 = \frac{10{,}400{,}000{,}000.}{3sh}$ | $H = 10$ in. $l^2 = \frac{800{,}000}{sh}$ | or $\frac{40{,}000}{l\sqrt{10h}}$ |
| $p = h$ divide numerator by 2 and solve for $l$ as before. | | For $p = h$ divide constant of numerator and denomination by 2 and solve for $l$ as before. |

4. BEAMS. SPACING OF STRUTS. 1 CU. FT. WET CONCRETE = 144 LB.

$d$ = Deflection. Assumed to be $\frac{1}{8}$ in.
$l$ = Spacing of Struts—inches.
$b$ = Width of assumed beam, inches.
$t$ = Thickness of lumber = 2 in.
$W$ = Width of concrete beam—inches.
$D$ = Depth of concrete beam—fnches.
$w$ = Weight per foot of length of concrete = $W(D+4)$.
$Wt$ = Total load on bottom form $b$ wide, $l$ long.

$$\therefore\ Wt = \frac{W\ (D+4)\ \ l}{12}$$

| For Deflection | For Strength |
|---|---|
| $d = \frac{Wl^3}{128EI}$ | $Wl = \frac{5}{3}\ bl^2f$ (1) |

SUBSTITUTING IN EQUATION (1).

| | |
|---|---|
| $l^4 = \frac{166{,}400{,}000}{w}\ b$ | $l^2 = \frac{96{,}000}{w}\ b$ |
| or | or (G) |
| $\frac{166{,}400{,}000}{W(D+4)}\ b$ | $\frac{96{,}000}{W(D+4)}\ b$ |
| For an inside form $b = W$ | |
| $\therefore\ l^4 = \frac{166{,}400{,}000}{D+4}$ | $l^2 = \frac{96{,}000}{D+4}$ |

5. FLOOR SLAB. SPACING OF FLOOR JOINTS. 1 CU. FT. WET CONCRETE = 144 LB.

$d$ = Deflection. Assumed to be $\frac{1}{32}$ in.
$l$ = Spacing of joists—inches.
$b$ = Width of assumed beam = 12 in.
$t$ = Thickness of sheathing = 1 in.
$T$ = Thickness of concrete slab—inches.
$w$ = Load per sq. ft. = 12 $(T+6)$.
$W$ = Total load on strip 12 in. wide, $l$ in. long.

$$\therefore W = \frac{w}{12} l = (T+6)l.$$

| For Deflection. | For Strength. | |
|---|---|---|
| $d = \frac{Wl^3}{128\ E\ I}$ | $Wl = \frac{3}{5}\ bt^2 f$ | (1) |

SUBSTITUTING IN EQUATION (1).

| $\therefore l^4 = \frac{5{,}200{,}000}{T+6}\ t^3$ | | $\therefore l^2 = \frac{24{,}000}{T+6}\ t^2$ | (G) |
|---|---|---|---|
| $t = 1$ in. | $l^4 = \frac{5{,}200{,}000}{T+6}$ | $t = 1$ in. | $l^2 = \frac{24{,}000}{T+6}$ |
| $t = 1\frac{1}{2}$ in. | $l^4 = \frac{17{,}550{,}000}{T+6}$ | $t = 1\frac{1}{2}$ in. | $l^2 = \frac{54{,}000}{T+6}$ |
| $t = 2$ in. | $l^4 = \frac{41{,}600{,}000}{T+6}$ | $t = 2$ in. | $l^2 = \frac{96{,}000}{T+6}$ |

## 6. SPACING OF GIRDERS.

$d$ = Deflection. Assumed to be $\frac{1}{32}$ in.
$l$ = Spacing of girders—inches.
$s$ = Spacing of floor joists—inches.
$b$ = Width of floor joist = 2 in.
$H$ = Depth of floor joists.
$w$ = Load per sq. ft. = $12(T+6)$.
$W$ = Total load on floor joist $l$ long spaced $s$ apart.

$$\therefore W = \frac{w}{144} ls = \frac{T+6}{12} ls$$

SUBSTITUTING IN EQUATION (1).

| $\therefore l^4 = \frac{10{,}400{,}000}{(T+6)s} H^3$ | | $\therefore l^2 = \frac{48{,}000\ H^2}{(T+6)s}$ | (G) |
|---|---|---|---|
| $H = 4$ in. | $l^4 = \frac{665{,}600{,}000}{(T+6)s}$ | $H = 4$ in. | $l^2 = \frac{768{,}000}{(T+6)s}$ |
| $H = 6$ in. | $l^4 = \frac{2{,}246{,}400{,}000}{(T+6)s}$ | $H = 6$ in. | $l^2 = \frac{1{,}728{,}000}{(T+6)s}$ |
| $H = 8$ in. | $l^4 = \frac{5{,}324{,}800{,}000}{(T+6)s}$ | $H = 8$ in. | $l^2 = \frac{3{,}072{,}000}{(T+6)s}$ |
| $H = 10$ in. | $l^4 = \frac{10{,}400{,}000{,}000}{(T+6)s}$ | $H = 10$ in. | $l^2 = \frac{4{,}800{,}000}{(T+6)s}$ |

### 7. SPACING OF STRUTS. 4 x 6 GIRDER.

$d$ = Deflection. Assumed to be $\frac{1}{17}$ in.
$l$ = Spacing of struts—inches.
$s$ = Spacing of girders—inches.
$b$ = Width of girder = 4 in.
$H$ = Depth of girder = 6 in.
$w$ = Load per sq. ft = $12(T+6)$.
$W$ = Total load on girder, spaced $s$ inches apart.

$$\therefore W = w\frac{sl}{144} = \frac{T+6}{12}ls$$

SUBSTITUTING IN EQUATION I.

| | |
|---|---|
| $\therefore l^4 = \frac{5{,}200{,}000}{(T+6)s}bH^3$ | $\therefore l^2 = \frac{24{,}000}{s(T+6)}bH^2$ (G) |
| $b=4$ in. $l^4 = \frac{20{,}800{,}000}{(T+6)s}H^3$ | $b=4$ in. $l^2 = \frac{96{,}000}{s(T+6)}H^2$ |
| $H=6$ in. $l^4 = \frac{4{,}492{,}800{,}000}{(T+6)s}$ | $H=6$ in. $l^2 = \frac{3{,}456{,}000}{s(T+6)}$ |

### 8. BEAMS.

Equating—

$$\frac{166{,}400{,}000}{W\,(D+4)}\,b = \left[\frac{96{,}000}{W\,(D+4)}\,b\right]^2$$

$$\therefore D = 55.4\,\frac{b}{W} - 4$$

For $b = W$

$D = 51.4$ in. (minimum) and $h = 41.62$ in

### 9. COLUMNS.

Equating; Weight per Cu. Ft. = 72 lb.

$$\frac{5{,}200{,}000}{3h}\,l^2 = \left[\frac{4{,}000}{h}\,l^2\right]^2$$

$$\therefore n = 9.23\,t \text{ and } l^2 = \frac{4{,}000}{9.23}\,t$$

| | | |
|---|---|---|
| $t = 1$ in. | $h = 9.23$ in. | $l = 20.8$ in. |
| $t = 1\frac{1}{2}$ in. | $h = 13.84$ in. | $l = 25.5$ in. |
| $t = 2$ in. | $h = 18.46$ in. | $l = 29.1$ in. |

Equating; Weight per cu. ft. = 144 lb.

$$\frac{2{,}600{,}000}{3h}\,l^2 = \left[\frac{2{,}000}{h}\,l^2\right]^2$$

$$\therefore h = 4.61 \text{ and } l^2 = \frac{2{,}000}{4.61}\,t$$

| | | |
|---|---|---|
| $t = 1$ in. | $h = 4.62$ in. | $l = 20.8$ in. |
| $t = 1\frac{1}{2}$ in. | $h = 6.93$ in. | $l = 25.5$ in. |
| $t = 2$ in. | $h = 9.22$ in. | $l = 29.1$ in. |

# REPORT OF COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

## Tests of Reinforced Concrete Columns.

### I. Preliminary.

Your Committee has made on behalf of the American Concrete Institute the herein reported tests of reinforced concrete columns of large size.

*Acknowledgments.*—In the execution of these tests the Institute has been favored with the co-operation of the United States Bureau of Standards in the use of the ten million pound compression testing machine at Pittsburgh, Pa., and of the Engineering Experiment Station of the University of Illinois in furnishing observers in conducting the tests. The tests were made possible through the active co-operation of those who donated the funds and materials for making and molding the columns.

The test columns were fabricated in New York by the Turner Construction Company. The making of the columns was inspected by A. S. Merrill, Assistant to the Chief Engineer of the Turner Construction Company, and D. A. Abrams, of the Engineering Experiment Station of the University of Illinois.

The tests of the columns and cylinders were made in the Bureau of Standards Laboratory at Pittsburgh. Especial acknowledgment is due S. W. Stratton, Director of the Bureau of Standards, P. H. Bates, in charge of the Pittsburgh laboratory and J. H. Griffith, Engineer-Physicist.

A. N. Talbot represented your committee; W. A. Slater and H. F. Gonnerman of the Engineering Experiment Station of the University of Illinois supervised the preparation of the columns for the tests and were in direct charge of the testing operations.

The extensive data taken on these tests have been reduced in the office of the Engineering Experiment Station of the University of Illinois, and in the office of the Turner Constructon Company.

The cement was furnished by the Atlas Portland Cement Company; it was sampled by Robert W. Hunt and Company. Two of the samples were tested by that company and a third by the Allentown Testing Laboratory.

The sand and gravel were furnished by the Phoenix Towing and Transportation Company and came from Hempstead Harbor, Long Island. Strength tests on 1–3 briquettes made up with the sand and Vulcanite Portland cement were made by Sanford E. Thompson; who also reported on the concrete making qualities of the aggregate.

The spiral reinforcement was furnished by the American Steel and Wire Company and the longitudinal reinforcement by the Carnegie Steel Company.

Also to F. E. Foss of Cooper Union, New York, for Laboratory facilities in making auxiliary tests.

## II. SCOPE OF TESTS.

Twenty-four columns were tested, having a variety of reinforcement, both longitudinal and lateral. In the tests, measurements were made to find the deformation in various parts of the column, including the stretch in the spiral reinforcement. This set of tests is a part of a program laid out by the committee for the purpose of determining the influence of both spiral reinforcement and longitudinal reinforcement upon the structural properties of columns. The schedule of tests as given in the Journal, Vol. I, No. 2, December, 1913, is here reproduced:

HOOPED COLUMNS ("Hooped" Applies to Columns With Spiral).

Group I. *Vertical steel the variable.*

| | | |
|---|---|---|
| Columns 1, 1a and 1b. | 1 per cent spiral. | No vertical steel. |
| Columns 2, 2a and 2b. | 1 per cent spiral. | 1 per cent vertical steel. |
| Columns 3, 3a and 3b. | 1 per cent spiral. | 2 per cent vertical steel. |
| Columns 4, 4a and 4b. | 1 per cent spiral. | 6 per cent vertical steel. |

Group II. *Effect of increasing spiral.*

| | | |
|---|---|---|
| Columns 5 and 5a. | 2 per cent spiral. | 1 per cent vertical steel. |

Group III. *Effect of protective shell.*

| | | |
|---|---|---|
| Columns 6 and 6a. | 1 per cent spiral. | 1 per cent vertical steel. |

Group IV. *Plain concrete.*

| | | |
|---|---|---|
| Columns 7 and 7a. | No spiral. | No vertical steel. |

VERTICALLY REINFORCED COLUMNS.

Group V.

Columns 8 and 8a. 1 per cent vertical steel. ¼-in. diameter ties, 12 in. apart.

Columns 9 and 9a. 2 per cent vertical steel. ¼-in. diameter ties, 12 in. apart.

Columns 10 and 10a. 4 per cent vertical steel. Double ¼-in. diameter ties, 12 in. apart.

The range of variables in the test columns is given in Table I.

Fig. 1 and 2 show the form and dimensions of the columns and the amount and position of the reinforcement. The shaft of the column for a length of 9 ft. 4 in. was generally about 21 in. in diameter. The two columns used to show the action of an outer protective coating were 24 in. in diameter. In the columns with spiral reinforcement, the four spacers had an area equal to about

TABLE I.—RANGE OF VARIABLES IN TEST COLUMNS.

| Spiral Reinforcement, Nominal per cent. | Longitudinal Reinforcement, per cent. | | | | |
|---|---|---|---|---|---|
| | 0 | 1 | 2 | 4 | 6 |
| 0 | 7, 7a | 8, 8a | 9, 9a | 10, 10a | |
| 1 | 1, 1a, 1b | 2, 2a, 2b 6, 6a | 3, 3a, 3b | | 4, 4a, 4b |
| 2 | | 5, 5a | | | |

0.2 per cent of the solumn section. This metal has not been considered in computing the percentages of longitudinal reinforcement. Ties ¼-in. in diameter were placed 12 in. apart in the columns without spiral reinforcement. They held the longitudinal reinforcement in position during the fabrication of the column.

The concrete was a 1 : 1½ : 3 mixture. With each column two cylinders 8 in. in diameter and 16 in. long were made for testing with the columns. Additional cylinders of the same size were made to determine the increase in the strength of the concrete with age.

## III. MATERIALS.

*Cement.*—The cement was taken from shipments received by the Turner Construction Company for use in the construction of the warehouse at which the test columns were made.

The amount passing a No. 100 sieve averaged 95.2 per cent for the three samples and that passing a No. 200 sieve averaged

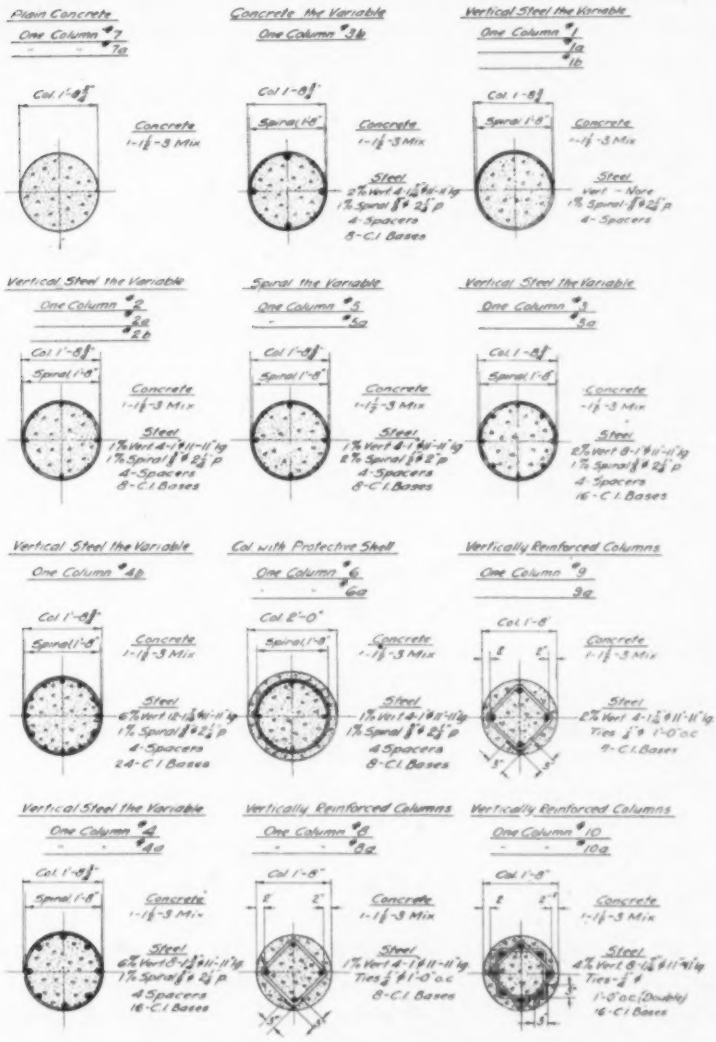



FIG. 2.—SECTIONS AND DETAILS OF TEST COLUMNS.

79.9 per cent. Table II gives the average tension tests for the three samples. In all cases the cement met the requirements of

the standard specifications of the American Society for Testing Materials (Standard No. 1 of the Institute).

*Sand and Gravel.*—The sand and gravel were taken from material delivered for use in the warehouse construction and were of good quality. The sand was siliceous, clean, hard and gritty. The stone contained some pebbles of feldspar and mica schist, but it was well graded and clean. The average results of the mechanical analysis of the sand and gravel are given in Tables III and IV.

The sand contained 35.5 per cent voids and had a specific gravity of 2.66. The ratio of the strength of these briquettes to

TABLE II.—TENSION TESTS OF CEMENT.

Each value for samples No. 1 and No. 3 is the average of five tests. For sample No. 2 each value is the average of three tests. Standard Ottawa sand was used in the 1:3 briquettes. The results are expressed in pounds per square inch.

| Sample No. | Neat. | | | 1-3. | |
|---|---|---|---|---|---|
| | 1 day. | 7 days. | 28 days. | 7 days. | 28 days. |
| 1 | 382 | 612 | 769 | 286 | 370 |
| 2 | 373 | 665 | 728 | 280 | 365 |
| 3 | 383 | 653 | 688 | 292 | 316 |
| Average | 379 | 643 | 728 | 286 | 350 |

those made with standard Ottawa sand was as follows: at age of 3 days, 1.51; at 7 days, 1.12; at 28 days, 0.98.

The gravel contained 34.4 per cent voids and had a specific gravity of 2.65.

*Reinforcement.*—The spiral reinforcement was bright Bessemer wire. During the time of winding the spirals coupons from 20 to 24 in. long were clipped for testing. Five coupons of each size ($\frac{1}{2}$ in. and $\frac{3}{8}$ in.) were tested. Table V gives a summary of the results of the tension tests. Fig. 3 gives the average stress deformation curves for the tests made at the University of Illinois. The elastic limit reported in the University of Illinois tests is Johnson's elastic limit. It is seen from Fig. 3 that at this stress there is considerable deviation from proportionality of stress to strain.

For the longitudinal reinforcement, three sizes of bar were used, 1 in., 1 $\frac{7}{16}$ in., and 1$\frac{3}{4}$ in. The bars were milled square on the ends to a length of 11 ft. 11 in. Four of the 1-in. bars were left over and these were used for physical tests.

Table V gives a summary of these tests and Fig. 4 gives a stress-deformation diagram for a tension test and a compression test of the longitudinal reinforcement. Compression tests were

TABLE III.—MECHANICAL ANALYSIS OF SAND.

| Sieve No. | Separation Size, inches. | Per cent Passing Average of Two Samples. |
|---|---|---|
| — | 0.5 | 100.0 |
| — | 0.25 | 99.7 |
| 10 | 0.073 | 91.0 |
| 20 | 0.034 | 68.8 |
| 30 | 0.022 | 45.9 |
| 40 | 0.015 | 28.9 |
| 50 | 0.011 | 15.0 |
| 80 | 0.007 | 5.2 |
| 100 | 0.0045 | 3.5 |
| 200 | 0.00275 | 1.8 |

TABLE IV.—MECHANICAL ANALYSIS OF GRAVEL.

| Size of Round Opening, inches. | Per cent Passing Average of Two Samples. |
|---|---|
| 1½ | 100.0 |
| 1¾ | 99.8 |
| 1 | 96.9 |
| ¾ | 83.0 |
| ½ | 55.7 |
| ¼ | 14.9 |
| ⅛ | 2.8 |

made on three of the coupons. Table VI gives the chemical analysis of the longitudinal reinforcement.

## IV. MAKING, TRANSPORTATION AND STORAGE.

In preparation for the work of making the columns, a 4-in. slab of concrete was built on the ground in a space in the warehouse of the Baltimore and Ohio Railroad in New York, which had been reserved for railroad tracks and which was inclosed on all but the south side. Care was taken to make this slab as nearly flat as possible. The column forms were set up on templates

of heavy paper on which were glued the cast iron base plates shown in Fig. 5, that located the longitudinal reinforcing bars. Similar cast iron plates were placed on top of the longitudinal reinforcement. The reinforcement was also wired to the forms.

TABLE V.—TENSION TESTS OF REINFORCEMENT.

| Sample No. | Diameter, in. | Turner Construction Co. Tests. | | | | University of Illinois Tests. | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Elastic Limit, lb. per sq. in. | Ultimate Strength, lb. per sq. in. | Elongation in 8 in., per cent. | Reduction of Area, per cent. | Elastic Limit ‡ lb. per sq. in. | Ultimate Strength lb. per sq. in. | Elongation in 8 in., per cent. | Reduction of Area, per cent. |
| 1 | ¾ | ..... | 84,600 | ... | 55.5 | 55,500 | 79,500 | 5.0 | 60.0 |
| 2 | " | ..... | 82,500 | 4.0 | 55.5 | 70,000 | 88,500 | 5.0 | 53.0 |
| 3 | " | ..... | 82,800 | 4.0 | 55.5 | 61,000 | 85,900 | 3.8 | 54.0 |
| 4 | " | ..... | 84,100 | ... | 54.7 | 66,000 | 88,000 | ... | 53.0 |
| 5 | " | 60,000 | 85,900 | 3.6 | 48.1 | ..... | ..... | ... | .... |
| 6 | " | 62,000 | 83,500 | 3.8 | 48.1 | ..... | ..... | ... | .... |
| Average | | 61,000 | 83,900 | 3.8 | 52.9 | 63,100 | 85,500 | 4.6 | 55.0 |
| 1 | ½ | ..... | 86,800 | ... | 44.5 | 58,500 | 89,600 | ... | 53.0 |
| 2 | " | ..... | 86,400 | 4.3 | 44.5 | 70,500 | 88,000 | ... | 51.0 |
| 3 | " | ..... | 85,100 | ... | 39.6 | 61,500 | 87,100 | 5.0 | 51.0 |
| 4 | " | 58,000 | 86,500 | 3.4 | 43.0 | 60,500 | 88,200 | 5.0 | 53.0 |
| 5 | " | 55,000 | 86,600 | 4.1 | 48.5 | ..... | ..... | ... | .... |
| Average | | 56,500 | 86,300 | 3.9 | 44.0 | 62,800 | 88,200 | 5.0 | 52.0 |
| 1 | 1 | 34,000 | 56,180 | 35.5 | 64.0 | 35,200 | 57,500 | 32.5 | 47.0 |
| 2 | " | 33,200 | 55,900 | 30.5 | 48.1 | 36,200 | 57,200 | 32.5 | 47.0 |
| 3 | " | 33,900 | 56,860 | 32.4 | 48.3 | 34,500† | ..... | .... | .... |
| 4 | " | 33,900 | 56,650 | 33.5 | 52.9 | 35,000† | ..... | .... | .... |
| 5 | " | 33,300 | 56,510 | 34.3 | 51.5 | 37,000† | ..... | .... | .... |
| Average | | 33,660 | 56,420 | 33.2 | 53.0 | 35,700 | 57,350 | 32.5 | 47.0 |
| * | 1 | 37,680 | 58,570 | 29.7 | 53.8 | ..... | ..... | .... | .... |
| * | 1⅛ | 37,430 | 59,400 | 30.7 | 51.1 | ..... | ..... | .... | .... |

* Carnegie Steel Co. tests.
† Compression tests, not included in the average.
‡ Johnson's Elastic Limit.

TABLE VI.—CHEMICAL ANALYSIS OF REINFORCING BARS.

| Size of Bar, inches. | Per cent. | | | | |
|---|---|---|---|---|---|
| | Carbon. | Manganese. | Phosphorus. | Sulphur. | Iron. |
| 1 | 0.18 | 0.41 | 0.012 | 0.033 | 99.375 |
| 1⅛ | 0.19 | 0.46 | 0.019 | 0.035 | 99.196 |

The forms were of wood, tight and well-made. The inside of the forms was brushed with oil. A quarter-section of the form at the base was left open until after the reinforcement was in place and inspected. Care was taken to see that the spacers

on the spiral reinforcement were secured in a vertical position. After sweeping out the loose pieces of tie wire and other refuse within the form, the opening was closed.

The plant for mixing and placing the concrete was the same as that used in the construction of the building and care was taken to obtain uniformity of conditions throughout. The workmen employed were those who had worked upon the construction

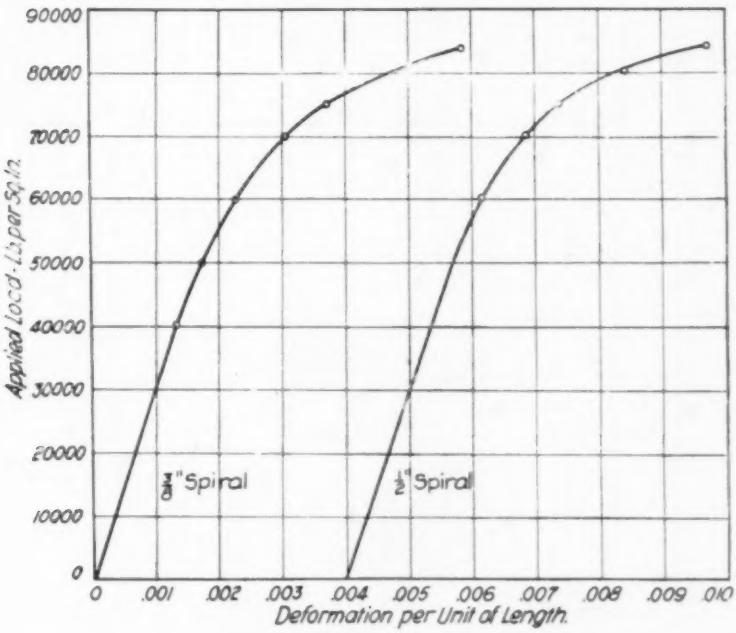



FIG. 3.—STRESS-DEFORMATION DIAGRAM FOR SPIRAL REINFORCEMENT. (AVERAGE OF 4 TESTS FOR EACH SIZE.)

of the building. A batch mixer of one-half yard capacity was used. Aggregates and mixing water were not heated. Each batch consisted of four bags of cement (which was taken as 4 cu. ft.), 6 cu. ft. of sand and 12 cu. ft. of gravel by loose measure. The sand and gravel were measured in two-wheel carts of 6 cu. ft. capacity. Before using, these carts were calibrated with a cubic foot box, and each load was stricken off as loaded. About seven pailfuls of water (200 lb.) were used for each batch. The mixer

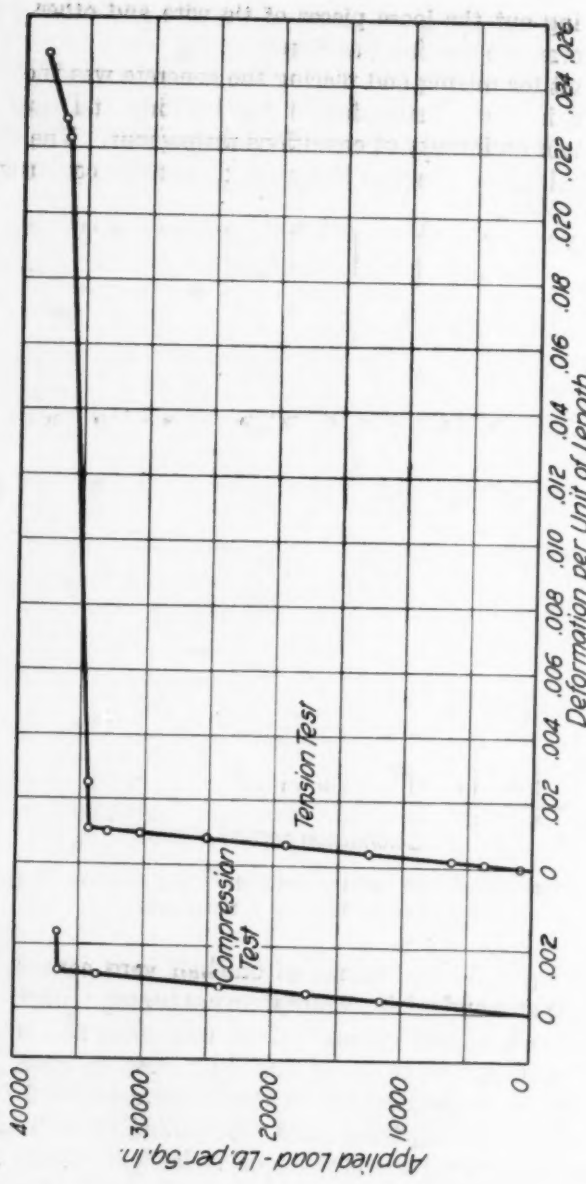



FIG. 4.—STRESS-DEFORMATION DIAGRAM FOR LONGITUDINAL REINFORCEMENT.

was run at 22 revolutions per minute. In general, each batch was mixed three minutes or more, the period depending upon the time taken to pour a batch into the column form. In general, each column required about two batches of concrete. After the concrete was poured into the forms, it was constantly poled in the manner used by the same gang of workmen in filling the concrete columns in the construction of the building. The column forms

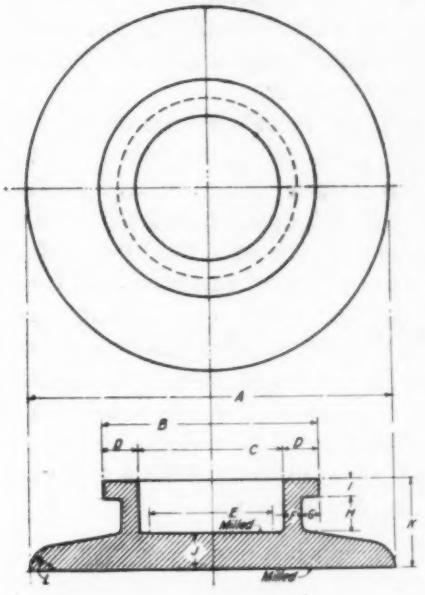



FIG. 5.—DETAIL OF CAST IRON BEARING PLATES.

| | | | |
|---|---|---|---|
| A–5 ″–3½″ | | G– ¼″– ½″ | |
| B–3 ″–2½″ | | H– ⅜″– ½″ | |
| C–2 ″–1½″ | | I – ½″– ¾″ | |
| D– ½″– ⅝″ | | J – ½″– ⅝″ | (exactly.) |
| E–1½″–1 ″ | | K–1½″–1½″ | |
| F– ¼″– ½″ | | L– ⅜″– ⅝″ | |

were used three times. They were removed after three days, except for the eight columns last filled, which were not removed until just before shipment.

Two test cylinders were made from separate batches of the concrete put into each column. The test cylinders were kept in the inclosure with the columns. They were wet down once in an effort to compensate for drying out more rapidly than the columns.

The temperature of the air on the days the columns were poured was as follows: November 19, 59 deg. F.; November 22, 56 deg. F.; November 26, 46 deg. F. In order to protect the columns from the freezing weather which came before they were shipped, they were surrounded and covered with tarpaulins and a salamander was kept burning in the inclosure to maintain a temperature of 50 deg. F., except during the time between December 1 and December 23. From December 1 to December 23 and also from January 3 to January 19 while the columns were being loaded and shipped to Pittsburgh the columns and cylinders were exposed to moderate winter weather.

The columns and cylinders were shipped to Pittsburgh on two flat cars. Care was taken to block the columns so that they could not shift in transit. They were seven days en route, and were unloaded without apparent damage. The columns were then stored on the testing floor of the Bureau of Standards laboratory. As the temperature of this room was frequently very low, especially at night, the columns were grouped close together and entirely inclosed by canvas. Gas was burned within the inclosure for several weeks, and a moderate temperature maintained. Taken in all, it is believed that the conditions for hardening in the 115 days which elapsed between making and testing were less favorable than may be expected with a temperature of 65 to 70 deg. F. maintained for 90 days.

## V. PREPARATION OF COLUMNS FOR TESTING.

The preparation of the columns for testing consisted in cutting into the concrete to expose the reinforcement to give opportunity for measuring the deformation in the reinforcement, setting steel plugs firmly in holes drilled in the concrete in the proper positions for the measurement of deformation of the concrete, and in the drilling of the gage holes.

For cutting the concrete, a tool driven by compressed air was used. The procedure in setting the plugs was as follows: The concrete in the vicinity of the gage holes was removed flush with the outer surface of the spiral. A steel plug was set flush with the outer surface of the longitudinal or spiral reinforcement. The method used is indicated in Fig. 6.

Fig. 7 shows the location of the gage lines upon the developed surface of a column and their numbering. There were slight

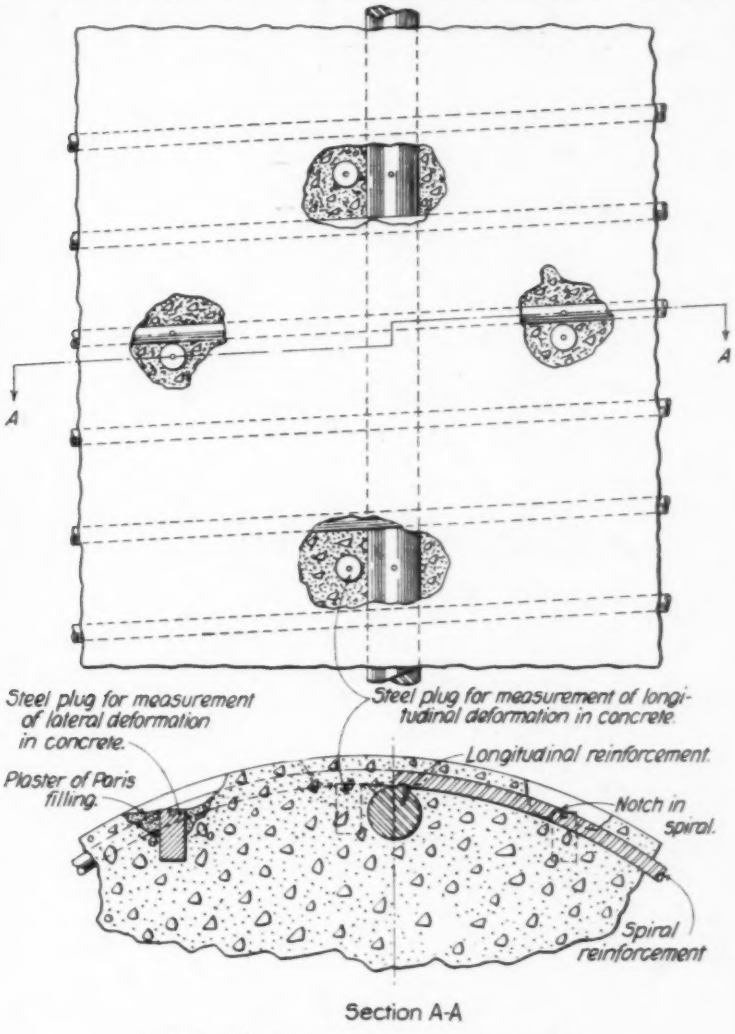



FIG. 6.—DETAIL OF LAYOUT OF GAGE LINES.

variations from this layout in individual columns. The number of gage lines varied from 40 to 112, but in most of the columns there were about 80.

## VI. TESTING.

The columns were tested in the 10,000,000-lb. testing machine of the Bureau of Standards laboratory at Pittsburgh, Pa. In this testing machine the load is applied by means of a hydraulic piston

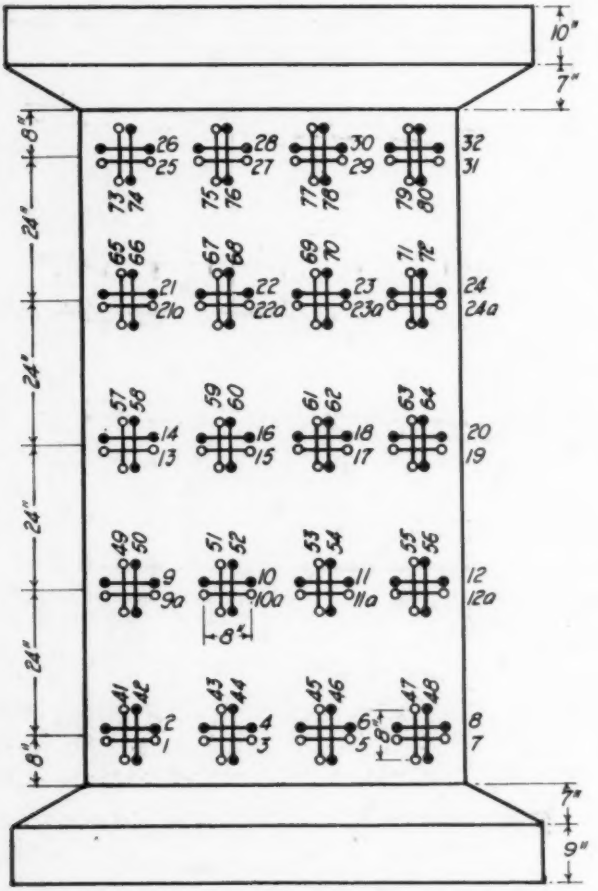



FIG. 7.—DEVELOPED SURFACE OF COLUMN, SHOWING LOCATION OF GAGE LINES.

which is actuated by a hydraulic pump. The load is weighed through a system of levers which are connected with the piston of a smaller hydraulic cylinder which communicates with the larger cylinder.

The columns were placed in position for testing after the gage lines had been prepared and numbered. A layer of plaster of Paris was used on the top and bottom surfaces of the columns. In placing the columns in the machine care was taken to have the upper surfaces of the column parallel with the upper platen of the testing machine and placed in such a way as to make external eccentricity of load a minimum. Any lateral thrust due to lack of uniformity in the action of the columns themselves was resisted by the four large main screws. The weighing beam of the testing machine was counterbalanced with the pump operating slowly. The pump was then stopped, the motor operating the top head was started, and the top head was brought down on the soft plaster of Paris at the top of the column until plaster of paris was forced out on all sides. An initial load of from 50,000 to 100,000 lb. was applied. The scale reading of the initial load was taken with the oil pump operating, and the piston moving very slowly.

The practice of the laboratory, under the instructions of the maker of the testing machine, is to balance the weighing beams while the pump is working, the purpose being to reduce the piston friction of rest to that of motion. The loads were read 2 minutes later and generally also 20 or more minutes later, but these readings were taken with the pump at rest and hence the conditions of the testing machine were somewhat different from those for which the first reading was taken.

After the column had been placed in the testing machine and the initial load had been applied, the initial strain-gage readings were taken. Berry strain-gages of the form in use at the University of Illinois were used.* At least two sets of initial readings were taken in all cases to serve as a check on each other. If the two sets showed serious disagreement in any gage line, a third set of readings was taken on these gage lines.

A full set of strain-gage readings was taken at each load. The time of taking a set of observations was twenty minutes, or more. From 6 to 15 sets of readings, corresponding to an equal number of loads, were taken in the test of a column, making from 250 to 1300 observations in all.

* For a description of this instrument and its use, see "The Testing of Reinforced Concrete Buildings under Load" in *Proceedings*, Vol. VIII, 1912, p. 168.

See also "The Use of the Strain Gage in the Testing of Materials," by W. A. Slater and H. F. Moore, Vol. XIII, *Proceedings*, Am. Soc. for Test. Mats., and "The Testing of Reinforced Concrete Buildings under Load," by A. N. Talbot and W. A. Slater, Bulletin 64, Univ. of Ill. Eng. Experiment Station.

The amount of load reported is the load indicated at the instant before stopping the pump of the testing machine. In any material having some plasticity a test piece under dead load will continue to shorten after the initial application of the load, or the test piece will be carrying less load if it is held at a constant length as is the case in a test of this kind. When a hooped column is loaded nearly to its ultimate capacity, it acts with some plasticity. To enable the extensometer readings to be taken under the same conditions over the whole column, the pump was kept at rest throughout the time of observation, but readings of the indicated load were taken at two minutes after the first reading and again at 20 minutes or more later. These readings are lower than the first reading, in some cases as much as 10 per cent less. How much of this decrease is due to the yielding of the column and how much to different conditions of weighing the load is not known. However, it is apparent that the real maximum load on a column under a rational interpretation of the application of the load would be somewhat less than the values here reported. It should be also added that the statement made in the discussion that the load held for a time, such as one minute, means that there was no great loss of load beyond the usual dropping of the indicated load, as found in the tests generally.

After the extensometer readings were taken at the maximum load, the pump of the testing machine was again put into operation in order to determine the manner of the final failure and to find how the column would act under continued deformation. The photographs which were taken show the columns as they appeared after having the deformation carried well beyond that produced by the maximum load. The spalling, which in the photographs appears to have removed much of the concrete outside the spiral, was much less pronounced at the maximum load.

## VII. DATA OF TESTS.

The compression strength of the concrete is given in Table VII.

The dates of making and testing the columns and cylinders are given in Table VIII. Table IX gives the ultimate loads carried by the columns and by the concrete cylinders, as well as general data of the test columns. Fig. 8 gives a typical stress

deformation diagram for the concrete cylinders. These cylinders were made with column 1. Stress-deformation diagrams are given in Figs. 00 to 00, plotted from the data taken in the tests.

TABLE VII.—COMPRESSIVE STRENGTH OF CONCRETE.

| Average age, days. | Compressive Strength, lb. per sq. in. | Remarks. |
|---|---|---|
| 7 | 872 | Average of 2, 8 x 16 in. cylinders. Air cured. |
| 28 | 1966 | Average of 6, 8 x 16 in. cylinders. Air cured. |
| 116 | 2943 | Average of 48, 8 x 16 in. cylinders. Air cured. |

TABLE VIII.—DATES OF MAKING AND TESTING COLUMNS AND CYLINDERS.

| Column No. | Date Made, 1913. | Date Tested, 1914. | Age, days. | Companion Cylinders. | |
|---|---|---|---|---|---|
| | | | | Date Tested. | Age, days. |
| 1 | November 19 | February 27 and 28 | 101 | March 3 | 104 |
| 1a | " 22 | March 11 | 109 | " 11 and 13 | 109, 111 |
| 1b | " 26 | " 25 | 119 | " 25 | 119 |
| 2 | November 19 | March 3 | 104 | March 2 | 103 |
| 2a | " 22 | " 16 | 114 | " 17 | 115 |
| 2b | " 26 | " 23 | 117 | " 23 | 117 |
| 3 | November 19 | March 4 | 105 | March 4 | 105 |
| 3a | " 22 | " 17 | 115 | " 17 | 115 |
| 3b | " 26 | " 26 | 120 | " 26 | 120 |
| 4 | November 19 | March 5 | 106 | March 5 | 106 |
| 4a | " 22 | " 18 and 19 | 117 | " 18 | 116 |
| 4b | " 26 | " 28 | 122 | " 27 and 28 | 121, 122 |
| 5 | November 19 | March 6 | 107 | March 6 | 107 |
| 5a | " 22 | " 20 | 118 | " 20 | 118 |
| 6 | November 19 | March 12 | 113 | March 13 | 114 |
| 6a | " 22 | " 19 | 117 | " 20 and 23 | 118, 121 |
| 7 | November 26 | March 27 | 121 | March 27 | 121 |
| 7a | " 26 | " 30 | 124 | " 31 | 125 |
| 8 | November 19 | March 10 | 111 | March 9 and 11 | 110, 112 |
| 8a | " 22 | " 23 | 121 | " 23 | 121 |
| 9 | November 19 | March 14 | 115 | March 9 and 14 | 110, 115 |
| 9a | " 22 | " 24 | 122 | " 24 | 122 |
| 10 | November 26 | March 31 | 125 | March 31 | 125 |
| 10a | " 26 | April 1 | 126 | April 1 | 126 |

the concrete and the full lines represent the measurements of The curves of column 3b, Fig. 9, may be considered typical. Elongation is plotted as positive and shortening as negative. The dotted lines represent measurements of the gage lines on

TABLE IX.—GENERAL DATA OF TESTS.

In columns with ½-in. spiral the pitch was 2 in.; in columns with ⅜-in. spiral the pitch was 2¼ in.

| Column No. | Section. | | Spiral Reinforcement. | | | Longitudinal Reinforcement. | | Maximum Load, lb. | Unit Load, lb. per sq. in. | Average Strength of 8 x 16 in. Cylinders, lb. per sq. in. |
|---|---|---|---|---|---|---|---|---|---|---|
| | Diameter, in. | Area, sq. in. | Kind. | Per cent. | Diameter, in., c. to c. | Kind. | Per cent. | | | |
| 1 | 20.84 | 341 | ⅜-in. | 0.94 | 19.95 | none | 0.00 | 1,621,500 | 4760 | 2868 |
| 1a | 20.82 | 340 | do. | 0.94 | 19.97 | do. | 0.00 | 1,555,000 | 4570 | 3083 |
| 1b | 20.86 | 341 | do. | 0.94 | 19.93 | do. | 0.00 | 1,774,000 | 5210 | 2885 |
| Average | | | | | | | | | **4850** | |
| 2 | 20.98 | 345 | ⅜-in. | 0.94 | 18.69 | 4 1-in. bars | 0.91 | 1,626,000 | 4710 | 3393 |
| 2a | 21.02 | 347 | do. | 0.93 | 19.68 | do. | 0.91 | 1,700,000 | 4900 | 2773 |
| 2b | 20.94 | 344 | do. | 0.94 | 19.82 | do. | 0.91 | 1,766,000 | 5140 | 2860 |
| Average | | | | | | | | | **4920** | |
| 3 | 20.95 | 344 | ⅜-in. | 0.94 | 19.37 | 8 1-in. bars | 1.83 | 1,737,500 | 5050 | 3185 |
| 3a | 20.96 | 344 | do. | 0.94 | 19.85 | do. | 1.83 | 1,647,000 | 4785 | 2575 |
| 3b | 20.94 | 344 | do. | 0.94 | 19.91 | 4 1⅞-in bars | 1.89 | 2,025,000 | 5885 | 2675 |
| Average | | | | | | | | | **5245** | |
| 4 | 20.66 | 335 | ⅜-in. | 0.95 | 19.53 | 8 1¾-in. bars | 5.75 | 2,069,000 | 6180 | 3095 |
| 4a | 20.88 | 342 | do. | 0.94 | 19.76 | do. | 5.63 | 1,968,000 | 5750 | 2670 |
| 4b | 20.85 | 341 | do. | 0.94 | 19.74 | 12 1⅞-in. bars | 5.71 | 2,165,000 | 6350 | 2775 |
| Average | | | | | | | | | **6090** | |
| 5 | 20.87 | 342 | ⅜-in. | 1.88 | 19.57 | 4 1-in. bars | 0.92 | 2,060,000 | 6025 | 3035 |
| 5a | 20.92 | 344 | do. | 1.88 | 19.73 | do. | 0.91 | 2,223,000 | 6495 | 2883 |
| Average | | | | | | | | | **6260** | |
| 6 | 24.09 | 455 | ⅜-in. | .... | ..... | 4 1-in. bars | .... | 1,740,000 | 3825 | 3130 |
| 6a | 24.23 | 462 | do. | .... | ..... | do. | .... | 1,720,000 | 3725 | 2768 |
| Average | | | | | | | | | **3775** | |
| 7 | 20.87 | 342 | none | 0.00 | ..... | none | 0.00 | 1,022,000 | 2990 | 2775 |
| 7a | 20.96 | 344 | do. | 0.00 | ..... | do. | 0.00 | 860,000 | 2500 | 2518 |
| Average | | | | | | | | | **2745** | |
| 8 | 20.04 | 315 | .... | .... | ..... | 4 1-in. bars | 1.00 | 1,040,000 | 3300 | 3308 |
| 8a | 20.31 | 324 | .... | .... | ..... | do. | 0.97 | 1,360,000 | 4200 | 3098 |
| Average | | | | | | | | | **3750** | |
| 9 | 20.13 | 318 | .... | .... | ..... | 4 1⅞-in. bars | 2.04 | 1,150,000 | 3620 | 3458 |
| 9a | 20.21 | 321 | .... | .... | ..... | do. | 2.02 | 1,206,000 | 3760 | 3055 |
| Average | | | | | | | | | **3690** | |
| 10 | 20.14 | 318 | .... | .... | ..... | 8 1⅞-in. bars | 4.08 | 1,451,000 | 4565 | 2835 |
| 10a | 20.16 | 319 | .... | .... | ..... | do. | 4.07 | 1,436,000 | 4500 | 2940 |
| Average | | | | | | | | | **4535** | |

the gage lines on the reinforcement. Fig. 9a is an enlarged stress deformation diagram giving the average measurements for the central plane shown in Fig. 9. Fig. 10 gives the distribution of deformations over the length of one column, 2b. This is a dia-

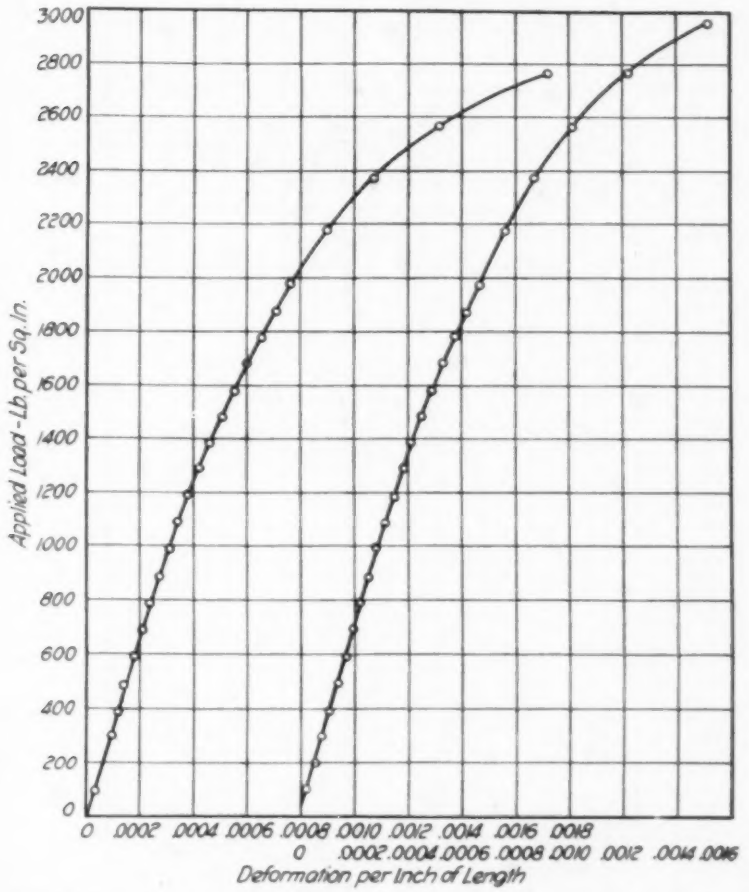



FIG. 8.—TYPICAL STRESS-DEFORMATION DIAGRAM OF CONCRETE CYLINDERS.

grammatic representation of the longitudinal compressive deformations and the lateral tensile deformations in the concrete. The measurements were taken at five planes, located 8 in. from the top and bottom and at three intermediate levels as shown on the

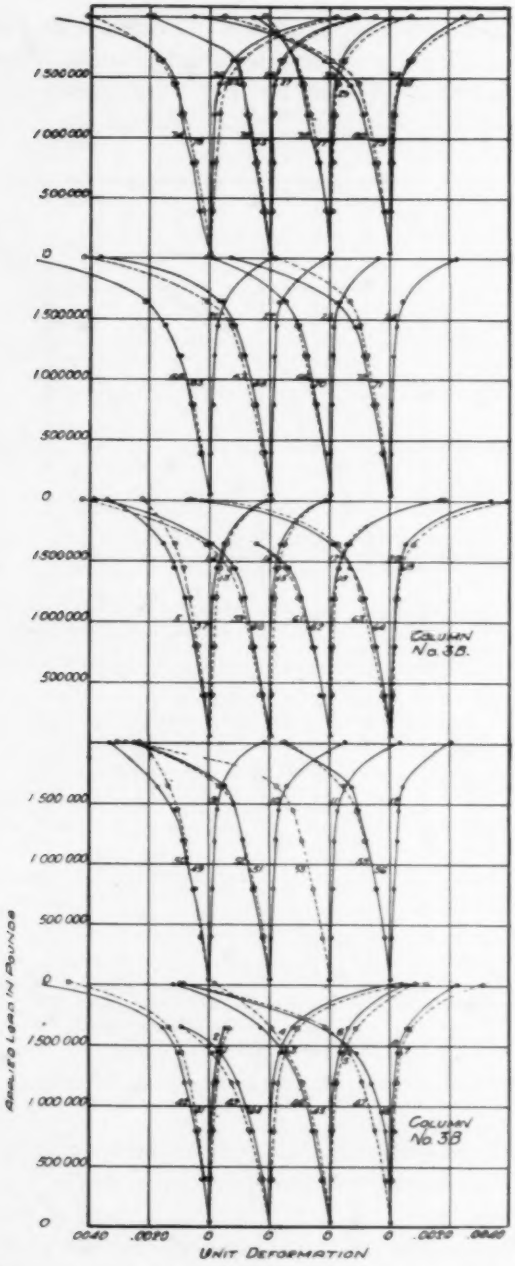



FIG. 9.—TYPICAL STRESS-DEFORMATION DIAGRAM, COLUMN 3B.

diagram. On the left are shown the measurements on one face and on the right those on the opposite face. In Fig. 11 and 12, the loads have been plotted according to the amount of longitudinal and spiral reinforcement.

Figs. 13 and 14 show some of the columns after failure. Fig. 15 gives details of spiral reinforcement failure.

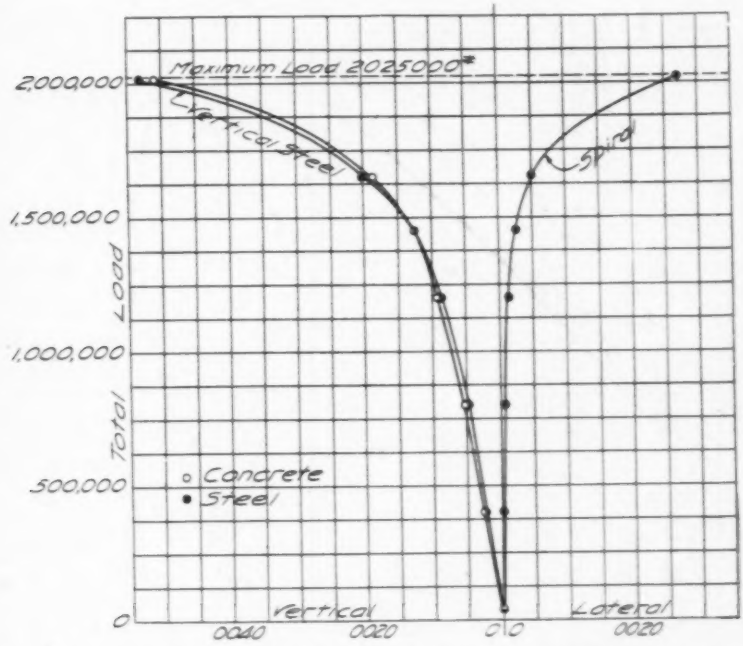



FIG. 9A. ENLARGED STRESS DEFORMATION DIAGRAM GIVING THE AVERAGE MEASUREMENT FOR THE CENTRAL PLANE SHOWN IN FIG. 9

## VIII. PLAIN CONCRETE COLUMNS.

The two plain concrete columns showed similar characteristics in the tests. No cracks or other signs of failure were visible until the maximum load was reached, except that in Column 7a a small longitudinal crack was seen in the shaft just below the head at a load of 750,000 lb. (2190 lb. per sq. in.). At the maximum load, longitudinal cracks developed all around the shaft of the column below the top of the head and extended down over the upper quarter of the length of the shaft. The load at once fell off 20

per cent or more, and with renewed movement of the pump, the load dropped still further. Column 7a and its test cylinders gave results which are lower than the average of all the test cylinders,

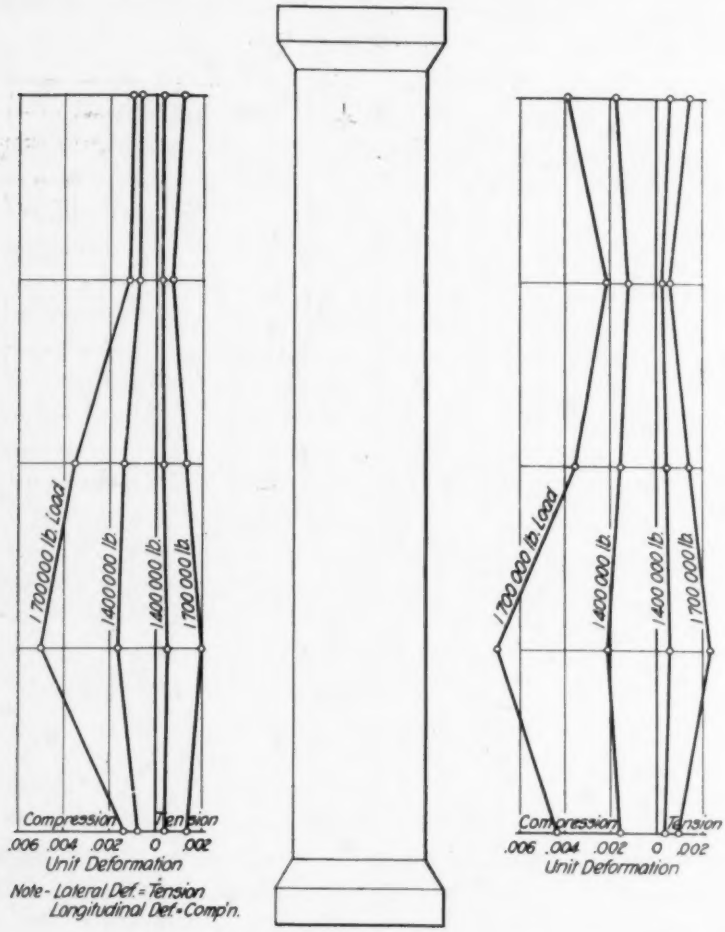



FIG. 10.—DISTRIBUTION OF LONGITUDINAL AND LATERAL DEFORMATION OVER LENGTH OF COLUMN FOR TWO LOADS. DIAGRAMS ON THE LEFT AND RIGHT ARE FOR OPPOSITE FACES.

and the action of 7a in the test indicates that the strength of this column was below what would be expected with concrete having the average quality of that in the other columns.

## IX. COLUMNS WITH LONGITUDINAL REINFORCEMENT AND NO SPIRAL REINFORCEMENT.

The series having longitudinal reinforcement and no spiral reinforcement includes two columns with 1 per cent of reinforcement, two columns with 2 per cent of reinforcement and two columns with about 4 per cent of reinforcement. Ties $\frac{1}{4}$ in. in diameter were placed 12 in. apart. In the tests there was no

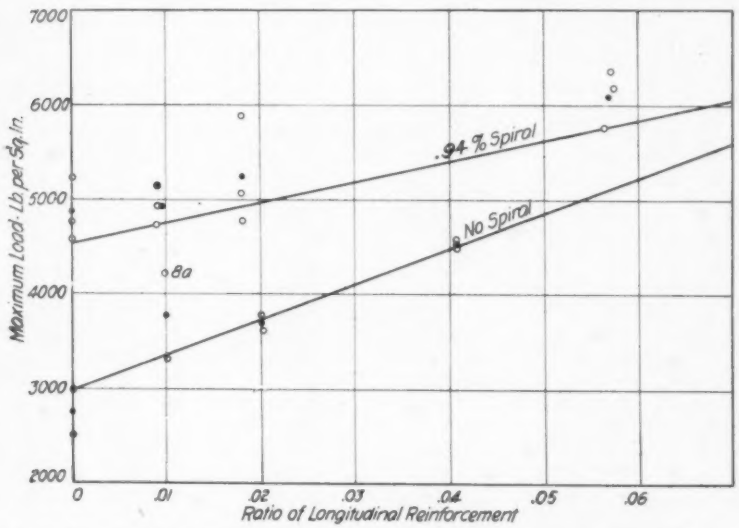



FIG. 11.—ULTIMATE LOADS—LONGITUDINAL REINFORCEMENT VARIABLE.

marked sign of distress before the maximum load was reached. In some cases no crack was visible below the maximum load. In other columns a minute crack was found at 85 to 90 per cent of the maximum load. Scaling of the longitudinal reinforcement at the gage holes where the bars were visible, was seen at loads below the maximum. In general the load fell off considerably a half-minute or so after the maximum load was applied.

The maximum loads carried by the columns are plotted in Fig. 11. The load taken by the longitudinal reinforcement is about 40,000 lb. per sq. in. The line shown in the diagram is expressed by the equation.

$$f = 3000\ (1-p) + 40000p$$

where $f$ is the average unit load over the whole column section, and $p$ is the ratio of the area of longitudinal reinforcement to the area of the column. This formula in effect assumes that at the maximum load the concrete carries 3000 lb. per sq. in. and the reinforcement 40,000 lb. per sq. in., making $n = 13\frac{1}{3}$.

## X. COLUMNS WITH SPIRAL REINFORCEMENT AND WITH AND WITHOUT LONGITUDINAL REINFORCEMENT.

The columns having 1 per cent of spiral reinforcement included columns with no longitudinal reinforcement and columns with nominally 1, 2 and 6 per cent of longitudinal reinforcement.

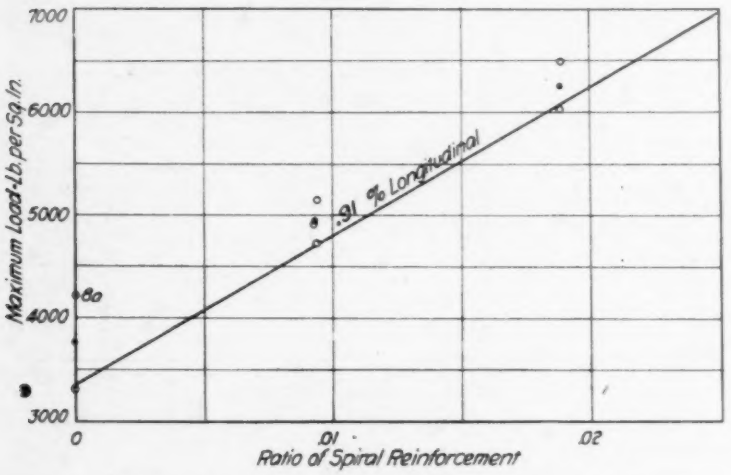



FIG. 12.—ULTIMATE LOADS—SPIRAL REINFORCEMENT VARIABLE.

Two columns contained 2 per cent of spiral reinforcement and 1 per cent of longitudinal reinforcement. The four spacers used to hold the spiral reinforcement in place had a sectional area equivalent to about 0.2 per cent of the column section, but this metal has not been considered in computing the percentages of longitudinal reinforcement.

The diagrams in Figs. 00 and 00 show indicated stresses in concrete, vertical reinforcement and spiral reinforcement, etc.

The load-deformation diagram of Column 3b given in Fig. 9 is typical of the hooped columns and will give an idea of the

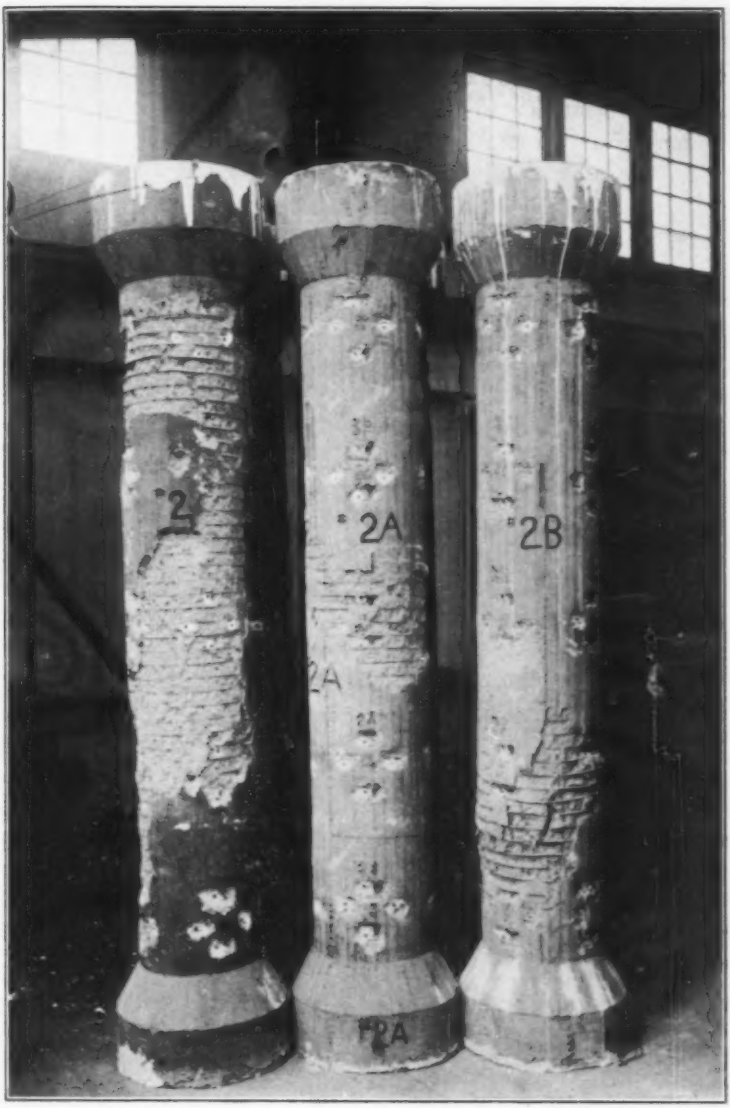



FIG. 13.—COLUMNS 2, 2A AND 2B AFTER FAILURE.

action of these columns. Both longitudinal and lateral deformations are shown. As has already been stated, elongation is plotted as positive and shortening as negative.

The action of the hooped columns during the early part of the tests was similar to that of the unhooped columns. At a load near the maximum load which was carried by the unhooped columns having an equal amount of longitudinal reinforcement, the observed stress in the spiral was, say from 6000 to 8000 lb. per sq. in., an amount which may be expected to correspond to the lateral expansion of the columns under slight restraint. The effect of this restraint at these loads was slight, perhaps equivalent to 150 lb. per sq. in. vertical pressure in the columns with 1 per cent spiral. The scaling of the longitudinal reinforcement where exposed at the gage holes was observed at loads similar to those observed for the unhooped columns.

Beyond the load which corresponds to the maximum load of an unhooped column the column shortened more rapidly with increase of load, the change in stiffness being very noticeable, and the stress in the spiral increased very rapidly. The decreasing stiffness of the column was very marked, and at loads near the maximum the ratio of the added load to the added longitudinal shortening was for columns with 1 per cent of spiral, say $\frac{1}{15}$ of that found at loads below the maximum load of an unhooped column. With continued increase of load the stretch of the spiral increased and beyond the yield point of the spiral reinforcement this stretch increased very rapidly relatively to the increment of load. As the maximum load was approached, the spiral stress was no longer uniform around the circumference, and it varied at different parts of the length of the column. Finally at some point the maximum strength of the spiral reinforcement was reached or the action of the column became very far from uniform, and the maximum load which the column would carry was attained.

There were few signs of distress or of approaching failure until near the maximum load. There were no spacing bars outside of the spiral to buckle. There was little or no spalling of the concrete outside the spiral, such as has been found in column tests when the coating outside the spiral was relatively much thinner. Fine vertical cracks usually appeared somewhat before the maxi-

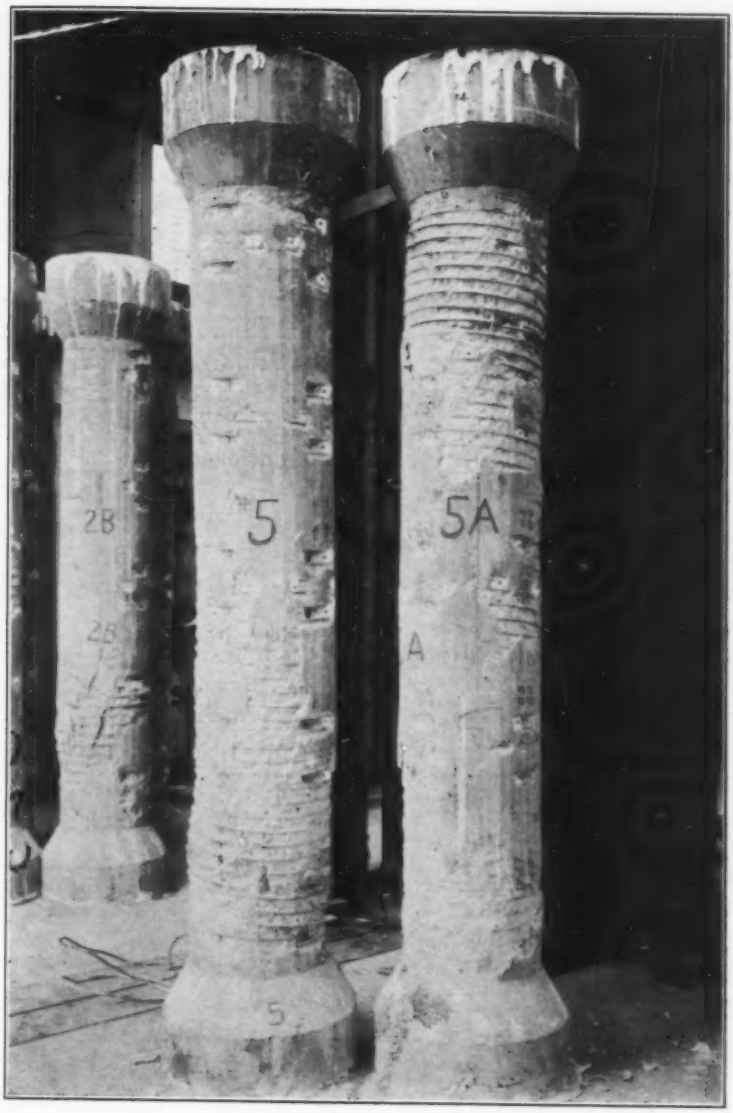



FIG. 14.—COLUMNS 5 AND 5A AFTER FAILURE.

mum load. Spalling occurred in one or more parts of the column—generally in several—either a little in advance of the maximum load or at the maximum load. The maximum load was held for some time, a minute or more, the pump operating during this time and the spiral reinforcement continuing to stretch. In many cases the spiral broke either at the maximum load, or at later and lower loads obtained by continuing to run the pump of the testing machine. The amount of shortening at the maximum load was generally about .004 in. per inch of length or more for 1 per cent spiral and .006 in. per inch for 2 per cent spiral.

In discussing the properties of the hooped columns, the question at once arises as to what part of the cross-section of the column is affected by the restraint of the spiral and whether the whole section acts to resist the load imposed on the column. The unit stress for any load will vary with the area of the cross-section used, and there will be a marked difference between the value obtained by using core section and that found with area of the entire cross-section of the column. The section which gives the proper interpretation of the action of the column is wanted in this study rather than the section which should be used in designing columns. Below the loads at which the spiral exerts an important effect, the conditions are similar to those of an unhooped column, the full cross-section should be used, and a proper interpretation will involve the use of unit stresses calculated on this basis. For loads which give marked stresses in the spiral it is obvious that all or nearly all of the area within the spiral will be fully restrained. It seems evident that an area outside the spiral may be acted upon similarly, the restraint being carried by the shearing and tensile resistance of the concrete. For the columns in question the space entirely outside the spiral amounted generally to about $\frac{3}{8}$ in., that is, a space equal to the diameter of the spiral reinforcement for the 1 per cent spirals. These columns showed no spalling up to nearly the maximum load. Calculations and diagrams made by using the core area only gave results which are inconsistent among themselves and not satisfactory in the interpretation of the data. The area of the gross section may be somewhat greater than the area which is effectively restrained, but it cannot be far from the right one. The unit stresses so found are smaller than would be obtained otherwise,

and the variation from the real effective section will partly make up for the possibility of having used for the ultimate loads values somewhat greater than may be warranted by the plastic quality which occurs in a hooped column near its maximum load and

FIG. 15.—METHOD OF FAILURE OF SPIRAL REINFORCEMENT.

when the spiral has been stressed beyond its yield point. It seems best, therefore, to use the gross area of the cross-section of the hooped columns. This should not be taken to have a bearing upon what section should be used in designing columns.

In Table IX the average unit loads given are based upon the

gross section of the columns. In Fig. 11 and Fig. 12 the plotted values are also based on the gross section.

From Fig. 11 it is apparent that the additional strength given by the longitudinal reinforcement is less in the case of columns with spiral reinforcement than in columns without spiral reinforcement. For loads beyond the resistance given in unhooped columns, it would seem that the longitudinal bars may offer little or no additional resistance, and that the lateral swelling of these bars may increase the stress produced in the spiral reinforcement without giving corresponding increase strength to the column.

In any study of the results attention may well be called to the abnormally high strength of column 8a, which is inconsistent with the other tests. Except for this column, the results are fairly consistent among themselves. The lines drawn on Fig. 11 and 12 represent the equation

$$f = 3000\,(1-p) + 40000\,p + 160000\,(1-10p)q.$$

where $f$ is the average unit stress over the whole column section, $p$ is the ratio of the area of the longitudinal reinforcement to the area of the column section, and $q$ is the ratio of the volume of the spiral reinforcement to the volume of the column. As the columns with 2 per cent spiral reinforcement contain only 1 per cent longitudinal reinforcement, there is nothing in the tests to show that this equation properly expresses the effect of longitudinal reinforcement in connection with 2 per cent spiral reinforcement. The formula, however, brings out a probable limitation on longitudinal reinforcement in hooped columns. Such a formula is limited by the range of the test variables, and it should be noted that the largest amount of spiral reinforcement in these tests is 2 per cent.

In discussing the results of the tests and in studying the diagrams in Fig. 11 and 12, it may be well to take into consideration the area of the spacers as affecting the percentages of longitudinal reinforcement and the applicability of the formula.

The strength given by spiral reinforcement is larger than has been found in other tests which have been reported previously. In judging the results of the tests it should be borne in mind that these are short columns (less than 6 diameters between heads), that the spiral reinforcement was well made and properly placed,

that the columns were straight, and that the loads were applied in such a way as to produce little bending moment in the columns. It has been shown by tests that irregularity of pitch and inclination of spiral produce the same effect as crookedness and may reduce the column strength.

In the discussion above, maximum loads are used and the values given are the ultimate strengths of the columns. A study of the load-stress diagrams made up from the data indicates that for any column the additional load due to the hooping is nearly proportional to the stress in the spiral reinforcement. This holds up to a stress of, say 85 per cent of the ultimate strength of the spiral reinforcement, when a marked variation in the stresses develops in different parts of a section. Rapid increase of stress at some point soon follows.

## XI. COLUMNS WITH FIREPROOFING LAYER.

The two columns with a thickness of about 2 in. of concrete outside the spiral were tested to determine the action of the fireproofing layer or protective shell under high loads. The action of the column through the first part of the test was similar to that of an unhooped column. At the maximum load the longitudinal deformations were about those of an unhooped column at its maximum load, somewhat less than the usual amount at most points of measurement, but reaching the usual amount at others. Up to the maximum load no cracks or other signs of distress were visible. At the maximum load the shell or layer outside the spiral rapidly cracked in various places in the upper portion of the column shaft, the load fell about 10 per cent and the fireproofing shell dropped off from this portion, either at the maximum load or a little later with a continued movement of the testing machine. With further movement of the head of the testing machine, and a corresponding shortening of the column, the load again increased and rose nearly to the original maximum load. During this time the spiral reinforcement in the exposed portion stretched as is usual with a hooped column, and the second maximum was that to be expected with a hooped column having a section corresponding to the outside diameter of the spiral.

It may be noted that the relation between the area of the gross section of a column having a given thickness of fireproofing,

and the area of the section inclosed by the spiral will depend upon the size of the column. For these columns, the load carried after the fireproofing layer failed was nearly as much as that which caused the failure of the outer shell. For larger columns (with the same thickness of fireproofing and same percentage of reinforcement) the maximum load carried by a hooped column would be larger than the load which would produce failure of the fireproofing layer. For smaller columns the reverse would be true.

## CONCLUSIONS.

The results of these tests warrant the following conclusions:

1. They seem to confirm the current practice as to the effect of varying percentages of longitudinal reinforcement within the range of these tests as expressed in the following formula:

$$f = f_c(1-p) + n f_c p$$

2. They indicate that columns with spiral reinforcement are stronger and tougher than columns with no spiral reinforcement.

3. For these tests it would appear that the spiral reinforcement is approximately four times as effective in giving ultimate strength to the column as the same volume of longitudinal reinforcement.

4. It appears that the effect of the longitudinal reinforcement decreases as the percentage of spiral reinforcement increases.

## PRELIMINARY
## REPORT OF COMMITTEE ON EDISON FIRE.

On Wednesday, December 9, 1914, at about 5.25 P. M., a fire occurred in the film inspecting building of the factory of Thomas A. Edison, Inc., West Orange, N. J., and rapidly spread to adjacent buildings of the plant, destroying six (one-, two- and three-story) wood and brick buildings, and burned out the contents of seven structures constituting three groups of reinforced concrete buildings.

Your Committee appointed to report on the Edison fire, after a careful investigation of the character of the buildings and their contents, the origin and intensity of the fire, and the behavior of the various materials of construction in the fire, have reached the following preliminary findings:

1. No building or any portion thereof can be considered absolutely fireproof. The resistance of buildings and building materials to fire is only relative; no material can resist a fire if it be of sufficient great intensity or duration.

2. The information obtained confirms the general principles already well established concerning protection against the origin and spread of fire.

3. The buildings of this plant were not in accord with present well established practice in that they deviated from this practice in the following particulars:

(a) Large undivided floor areas. (Lack of fire walls and stops.)

(b) Unprotected exterior openings. (Wooden sash with plain glass, etc.)

(c) Absence of proper fire walls, partitions and fire doors.

(d) Failure to properly inclose vertical openings, stairway and elevator shafts.

(e) Absence of adequate sprinkler equipment.

(f) Absence of adequate water supply and fire fighting apparatus.

(g) Failure to either isolate non-fireproof buildings or to protect against them.

(h) Failure to isolate hazardous processes of manufacture either by placing such processes in separate buildings or surrounding them by proper fire walls.

4. The fire fully demonstrated the advantages of monolithic structures. The fact that at five different places several of the wall columns were rendered useless and yet the upper portions of the building stood intact, is evidence of the superior merits of concrete in monolithic construction.

5. Considering the extraordinary conditions surrounding this fire, the behavior of the concrete buildings was highly satisfactory and constitutes an excellent demonstration of the merits of concrete as a fireresisting building material. It is not so surprising that the concrete buildings were damaged as that any material should have so satisfactorily withstood these unusual conditions.

6. This fire brought out the following important problems which are considered worthy of further investigation:

(a) The strength of the structure after the fire; to be determined by floor load tests.

(b) The extent to which the floor system has been strengthened by the method of reconstruction.

(c) The strength of the columns after the fire; the strength of the restored columns.

(d) The effect of expansion, from temperature changes, on large units.

(e) Such other effects of this fire on the structures and structural materials as the conditions may permit.

7. Your Committee is of the opinion:

(a) That the best type of construction is one which eliminates corners and presents flat or rounded surfaces to the action of the fire. That interior columns of circular or octagonal section offer greater resistance to fire and are less liable to damage than columns of square section, and that the elimination of sharp corners in the floor construction is desirable.

(b) That good construction requires a sufficient number of lateral ties or hooping in the columns, and that such construction would offer greater resistance to fire.

(c) That in the beam and girder type of construction in order to fully develop T action the slab should be cast at the same time that the beams and girders are poured, and there should be adequate ties between the reinforcement of the beams and girders and that of the slab.

The detailed information upon which the foregoing statements are based is submitted.

A series of investigations and tests are being arranged to develop additional data which, it is expected, will require some time and which will be presented in a final report as soon as the data can be obtained and collated.

Your Committee wishes to express its admiration for the commendable energy displayed by Thomas A. Edison and his associates in the rehabilitation of this plant. Two of the buildings are already occupied, a number of the other structures replaced with temporary buildings, and work is actively in progress for the restoration of the remaining portions of the plant.

Your Committee desires to acknowledge its obligations and hereby extends its thanks for the courtesies extended to it and the assistance rendered in its investigations by the officials of Thomas A. Edison, Inc., The Moyer Engineering and Construction Company, and T. L. Condron and Company, Engineers.

Respectfully submitted,

CASS GILBERT, *Chairman.*
WALTER COOK
W. H. HAM
RICHARD L. HUMPHREY
RUDOLPH P. MILLER
C. L. NORTON
J. KNOX TAYLOR
E. J. MOORE, *Secretary.*

February 3, 1915.

## DETAILED INFORMATION SUBMITTED BY COMMITTEE ON EDISON FIRE.

### 1. History of the Reinforced Concrete Buildings.

The reinforced concrete buildings within the burned area (Figs. 1-6) were constructed during 1905, 1906 and 1907, in the following order:—Numbers 7, 15, 25, 13, 24, 12 and 11.

These buildings are of the beam and girder type; the columns are square or rectangular in section with the exception of

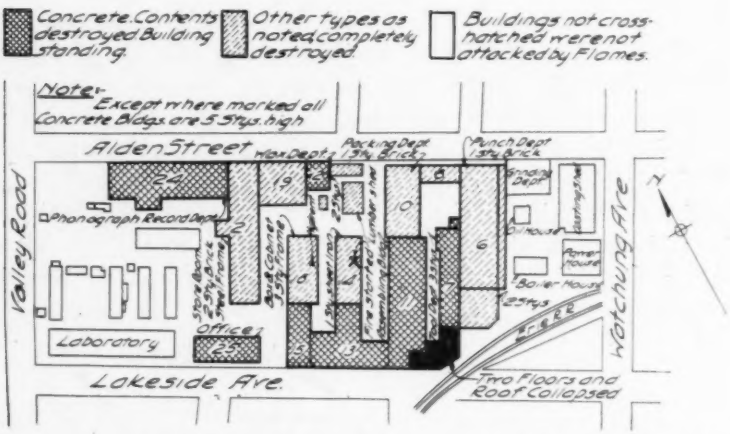

Concrete. Contents destroyed Building standing.
Other types as noted completely destroyed
Buildings not cross-hatched were not attacked by Flames.
Note:- Except where marked all Concrete Bldgs. are 5 Stys. high
Alden Street
Valley Road
Hatching Ave.
Lakeside Ave.
Laboratory
Office
Two Floors and Roof Collapsed

FIG. 1.—GENERAL PLAN OF THE PLANT OF THE THOMAS A. EDISON, INC., WEST ORANGE, N. J., SHOWING LOCATION OF BURNED AREA AND TYPE OF BUILDINGS AFFECTED.

those for the first and second stories of Building No. 7, which are circular; the walls have the usual window openings extending from column to column (the wall beams being cast with the floors and the curtain walls afterwards). The interior partitions were constructed of 3 and 4-in. plaster of Paris cinder block, plastered with Portland cement mortar. All window frames and sash were of wood except those in the monitors of Buildings Nos. 11 and 24, which were of metal; no wired glass was used. The interior metal covered wood doors did not fulfill the present re-

quirements for proper fire doors. For typical details of reinforced concrete buildings see Figs. 7-14, inclusive.

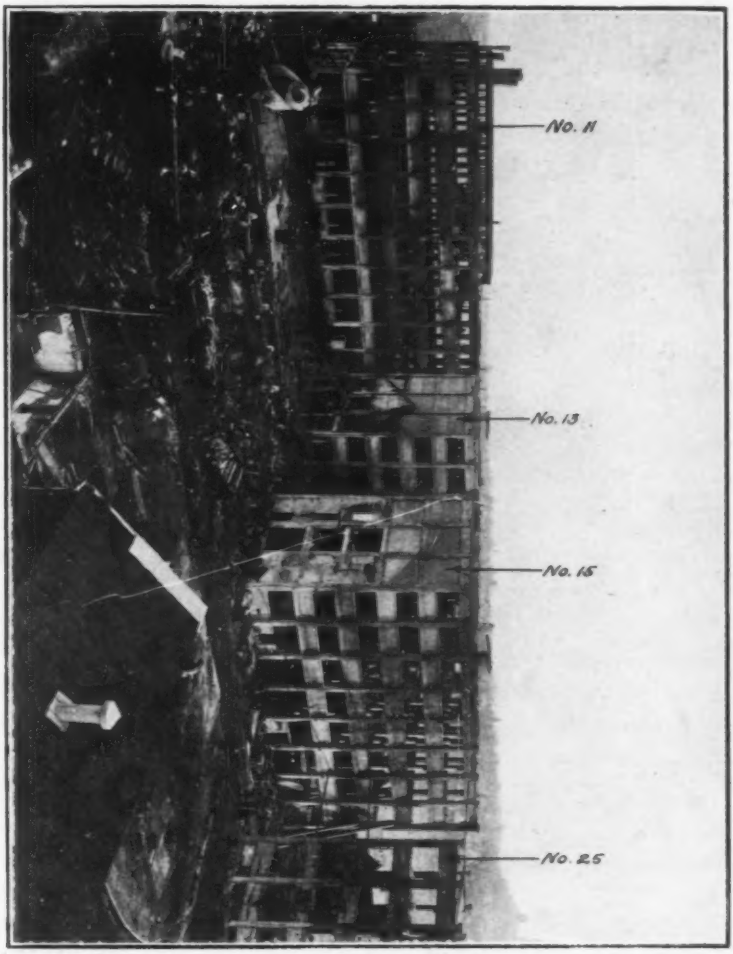



FIG. 3.—BUILDINGS NOS. 11, 13, 15, 25 AFTER THE FIRE. ATTENTION IS PARTICULARLY CALLED TO THE REINFORCED CONCRETE CURTAIN WALLS IN THE ENDS OF THE BUILDINGS, NOS. 13 AND 15 ABOVE THE ROOF OF THE ADJACENT BUILDINGS WHICH WERE COMPLETELY DESTROYED.

## 2. FIRE PROTECTION.

The water supply was obtained from a 6-in. line from the Orange Water Works, from an 8-in. line from the West Orange

Water Works, a 6-in. low pressure street line surrounding the plant from West Orange Water Works, and from the artesian wells of Thomas A. Edison, Inc., distributed through a 14-in. main. There was a 6-in. connection from the Orange Water Works' 6-in. line, a 4-in. from the West Orange 8-in. line and one 6-in. and two 4-in. connections with the 6-in. street line. There were two 800 gallon centrifugal pumps and one 250 gallon steam pump. The only sprinkler installation in the burned area was in Building No. 18 (Fig. 1), which is said not to have been in accordance

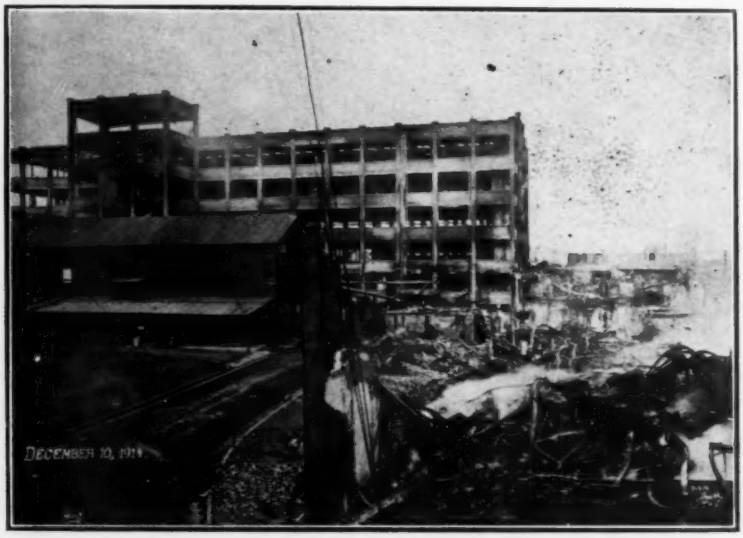



FIG. 4.—SOUTH FRONT OF BUILDING NO. 24 AFTER THE FIRE.

with present day requirements. All the buildings were supplied with chemical fire extinguishers. There were standpipes distributed throughout the plant, and several fire hydrants in the yard with hose equipment. The local fire department was handicapped through lack of a sufficient water supply.

### 3. CHARACTER OF THE CONTENTS OF THE BUILDINGS.

Diagram 1 gives in detail the nature of the contents of the different buildings at the time of the fire.

4. ORIGIN AND PROGRESS OF THE FIRE.

The fire started in one of the film inspection booths in the one-story wood frame Building No. 14 (Fig. 1) at about 5.25 P. M. At this time the wind was blowing from a point a little

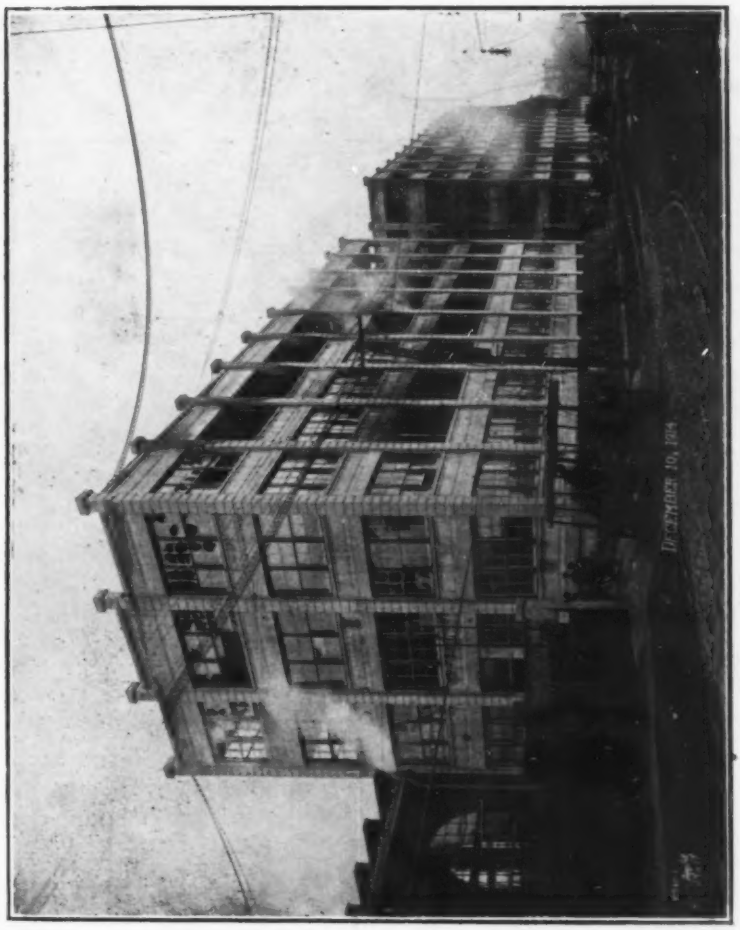


FIG. 5.—BUILDINGS NOS. 15 AND 25 FROM LAKESIDE AVENUE AFTER THE FIRE.

east of south, so that the fire quickly gained headway through the film stock and on the wood frame of the building, it spread to the lumber piles on the north and the three-story frame, box and cabinet making building (No. 18) on the west. There was a large

stock of unfinished cabinets, bundles of wood veneer, etc., in Building No. 18 and the lower stories of Building No. 15, so that the fire by 6 or 6.30 P. M. had reached serious proportions. The brick walls of Buildings Nos. 19 and 21 and the concrete walls of Building No. 12 seem to have checked the fire for a time in the region of Building No. 18. The exterior and interior walls and partitions in the first and second stories of Buildings Nos. 13 and 15 (Fig. 19) were at first effective in resisting the progress of the

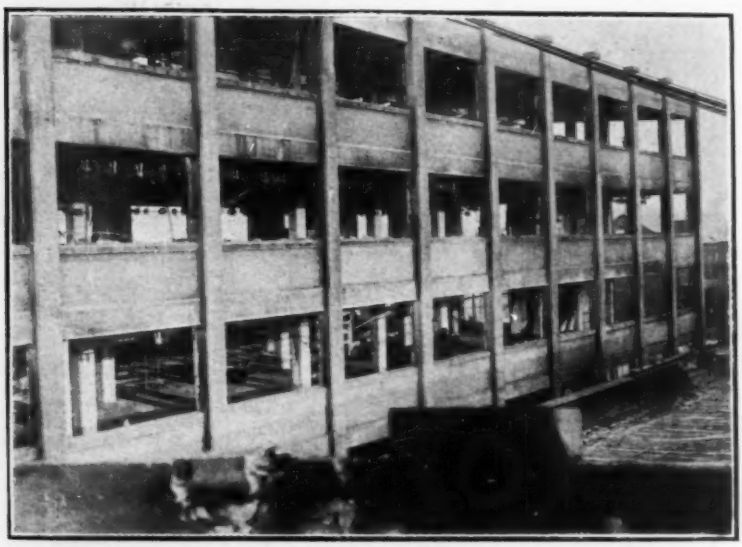


FIG. 6.—THREE UPPER FLOORS OF BUILDING NO. 11, GIVING AN IDEA OF THE EXCELLENT CONDITION OF THIS STRUCTURE AFTER THE FIRE.

fire, but it spread rapidly through windows to the third, fourth and fifth stories of these buildings, which presented an undivided area, and traveled east against the wind through the Lakeside Avenue ends of these buildings. At 7 o'clock the fire had entered the Lakeside Avenue end of Building No. 11 in the three upper stories and continued through to the east end of Building No. 11 and then north through this building. A section of three upper stories of this building, which seems to have been subjected to fire not more than 1½ hours, collapsed at 8.30. At 9 o'clock the fire reached its height in the three upper stories of the main

portion of Building No. 11. (See Fig. 15.) In the meantime, the finished cabinet stock in the first story of the same building had caught fire from the adjoining buildings on the west and the fire spread through to Building No. 10 and thence up the temporary

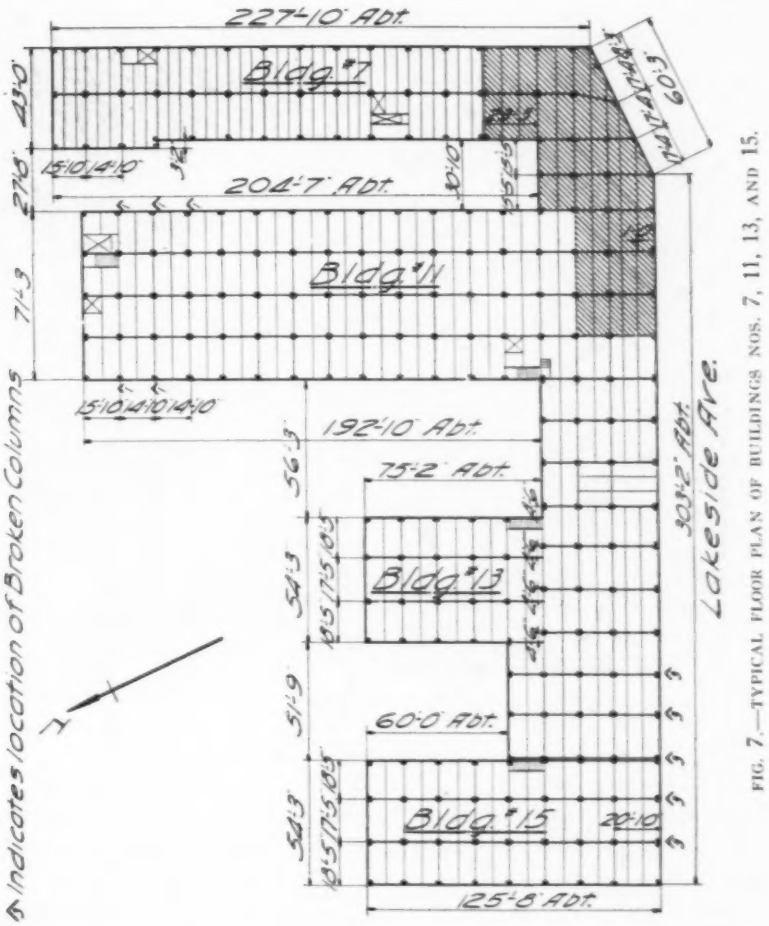



FIG. 7.—TYPICAL FLOOR PLAN OF BUILDINGS NOS. 7, 11, 13, AND 15.

attacked by the fire from both directions. At this time the roof wood enclosure at the north end of Building No. 11, and by 9 o'clock the two northerly bays of this building were burning brightly in all stories and a section near the center was being

of Building No. 10 had collapsed and the fire was traveling against the wind along the roof of Building No. 8, on Alden Street, towards Building No. 6. The fire did not reach Building No. 7 until between 9 and 10 o'clock, and the roof of Building No. 6 did not fall until after 10 o'clock. At about 9 o'clock the fire had started in the second floor of the Lakeside Avenue end of Building No. 11, adjacent to Building No. 13. The fire in the first story of Building No. 11, in the Lakeside Avenue

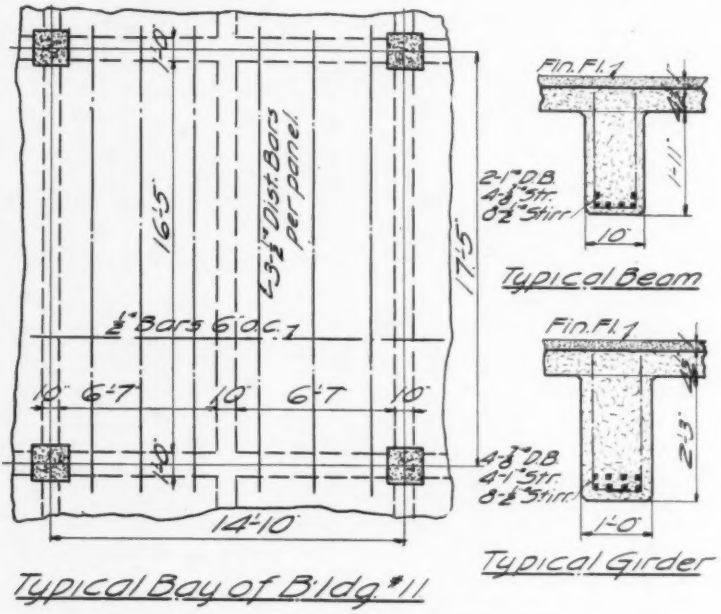



FIG. 8.—TYPICAL FLOOR DETAILS OF BUILDING NO. 11.

end, did not start until after 10 o'clock; the indications are that the damage in this story was due to the exceedingly hot fire from the burning of the adjacent portion of Building No. 6, which had a wood frame in the second story, and was used for film manufacture. This was connected to third story of Building No. 7 by an inclined passageway.

Mr. Edison gave the Committee the following explanation of the collapse of the east end of Building No. 11:—On the fourth floor of Building No. 11 were large wooden plating vats con-

taining nitric, hydrochloric and sulphuric acids; as these vasts were consumed by the fire the acids were released, combining to

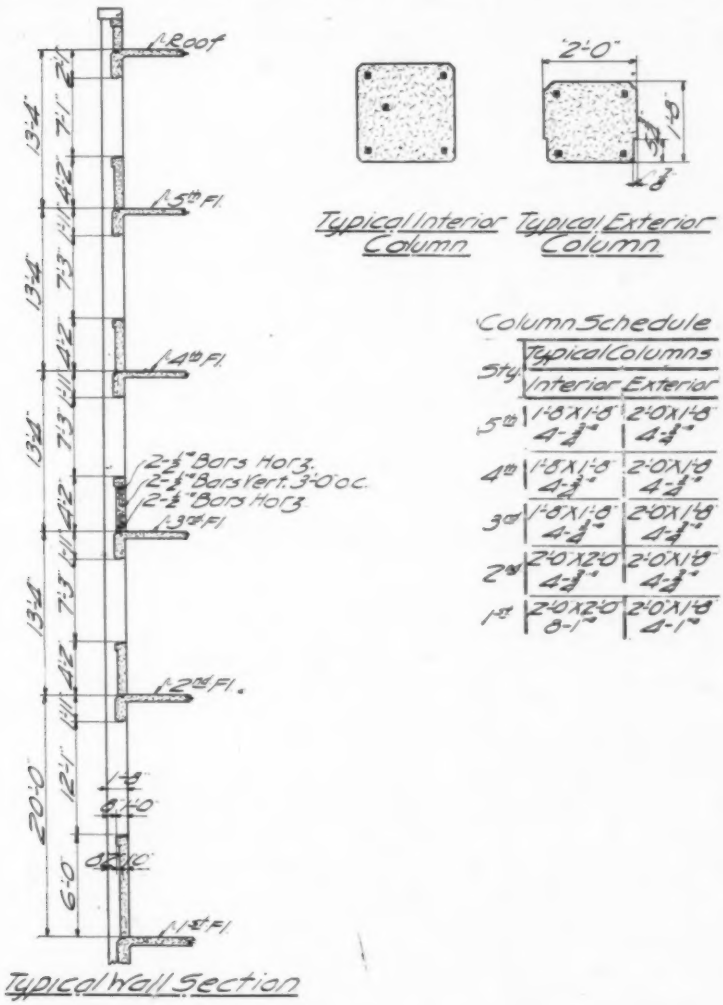



FIG. 9.—TYPICAL WALL AND COLUMN DETAILS OF BUILDING NO. 11.

form aqua regia, and attacked the concrete of the columns below, causing their collapse.

At the other end of the burned area the progress of the fire

was at first more rapid; by 7 o'clock it was general in Buildings Nos. 19 and 20; by 8 o'clock had destroyed these buildings, and was at its height in the top story of Building No. 24, but was being held at the east end in the lower stories of this building. (Figs. 16 and 17.) After reaching the west end of the top story of

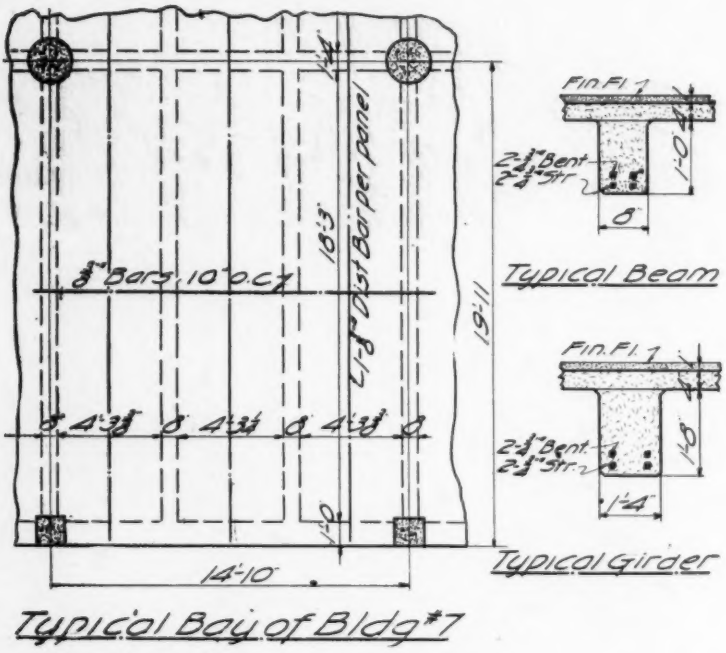



FIG. 10.—TYPICAL FLOOR DETAILS OF BUILDING NO. 1.

this building the fire traveled to the lower stories down the temporary wooden encosure and worked back to the fire coming through the lower stories of the building. By 9.30 the temporary west end enclosure had burned (Fig. 18), and by 10.30 there was comparatively little fire in this building except in the second story and at the extreme west end of first story. The office building (No. 25) caught fire at the extreme east end about 7 o'clock, and was not extinguished until 11 o'clock. The principal efforts by the fire department were directed to this building in order to prevent the fire spreading to the Laboratory, on the west,

and the storage battery buildings, on the opposite side of Lakeside Avenue.

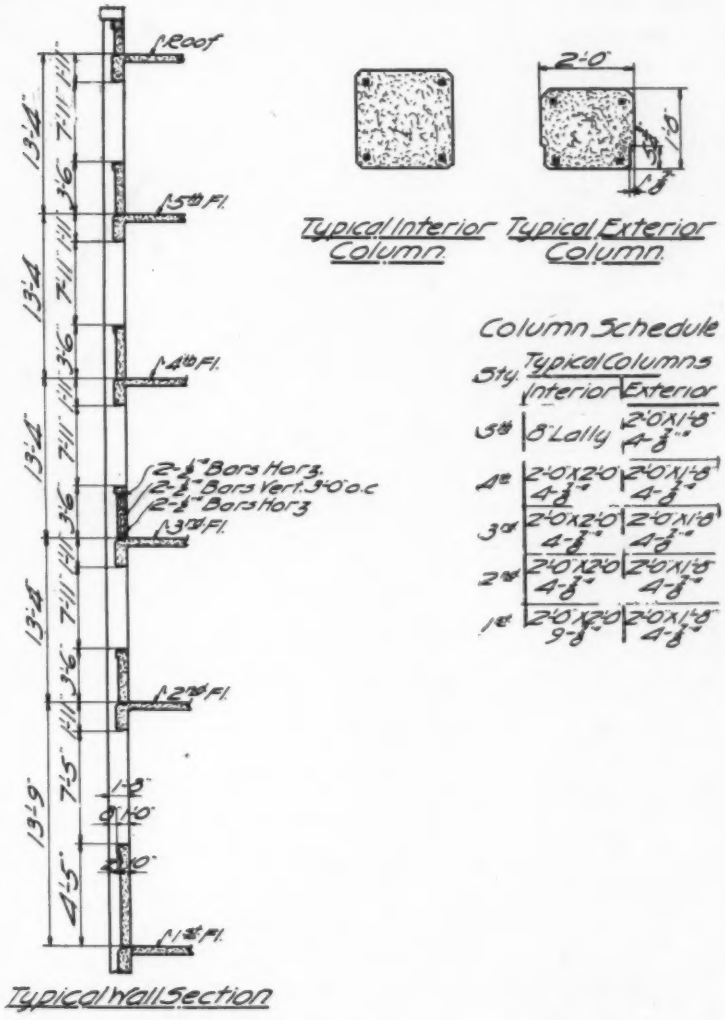



FIG. 11.—TYPICAL WALL AND COLUMN DETAILS OF BUILDING NO. 7.

## 5. EFFORTS TO CONTROL.

The local fire department first had a stream of water from Lakeside Avenue on the first and second stories of Building No. 15; then a stream on the west side of the first story of Building

No. 11 from a hose line through the driveway from Lakeside Avenue. Subsequently two streams were operated from the Erie Railroad tracks, on this and adjacent buildings; two streams

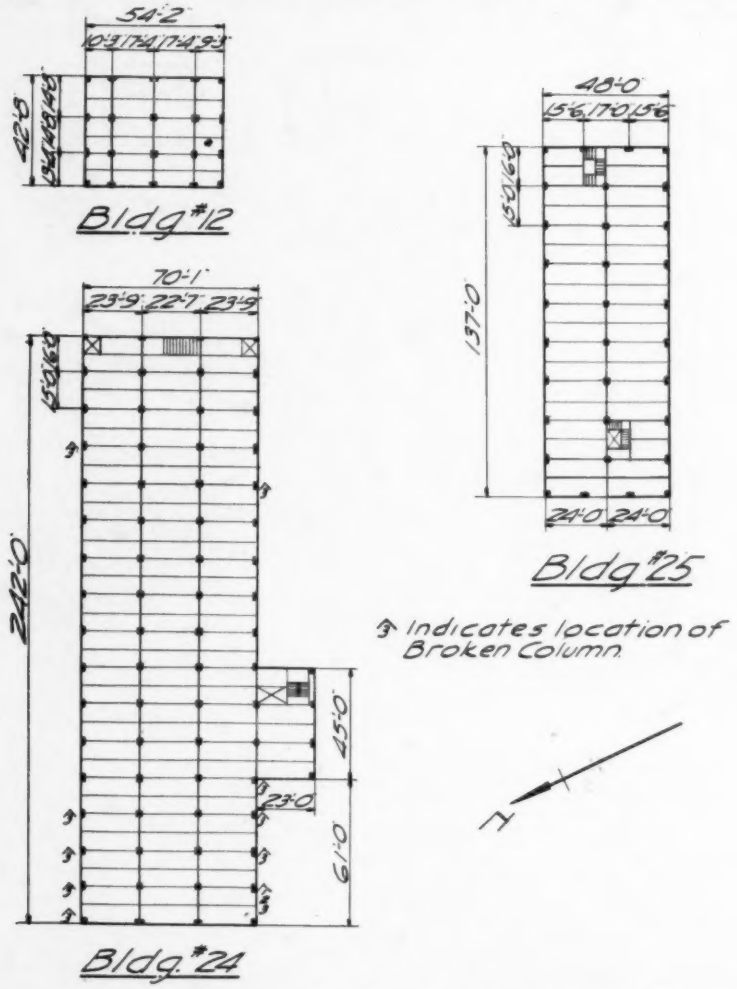



FIG. 12.—TYPICAL FLOOR PLAN OF BUILDINGS NOS. 12, 24 AND 25.

were directed against the office building No. 25—one operated from Lakeside Avenue and the other from the roof of the Laboratory boiler house. On Alden Street two streams were in

operation—one at or near each end of Building No. 24. The plant force, during the early progress of the fire, succeeded in retarding it in Building No. 21, in all stories of Building No. 24 except the top, and later prevented its spread westerly to the several small structures adjoining the main Laboratory buildings.

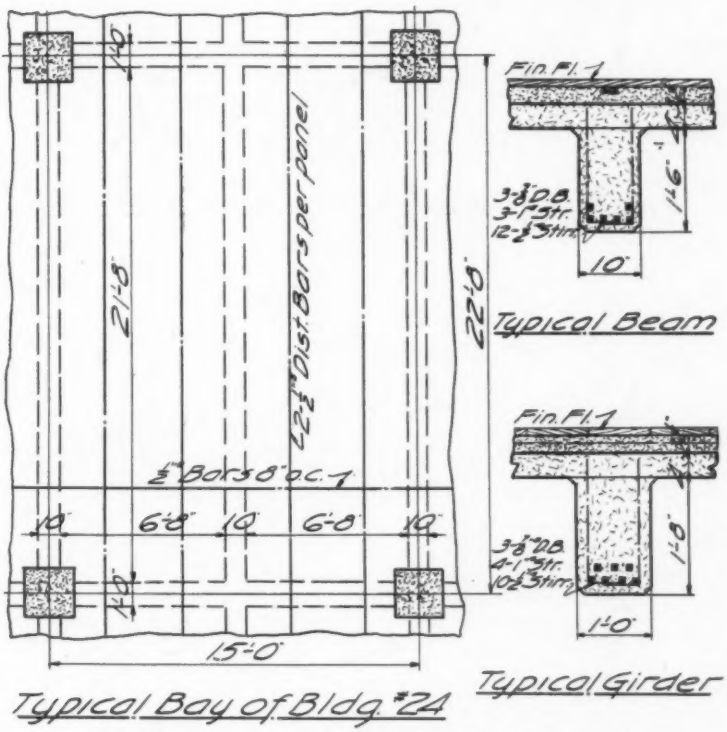



FIG. 13.—TYPICAL FLOOR DETAILS OF BUILDING NO. 24.

## 6. SEVERITY OF THE FIRE.

Melted copper and brass were found in nearly all of the buildings reached by the fire, and several pieces of fused iron were also found. In nearly all of the concrete buildings melted glass was found on the window sills. The concrete in the wall columns was less damaged by spalling than that in the interior columns and the ceiling beams, which would indicate that the heat in the interior of different stories was much greater than near the

walls. The hottest fire and the highest temperature were apparently reached in the three-story frame building (No. 18), where the top of the iron hydrant, near the east side, was melted. During the burning of this building the critical point in the progress of the fire was reached, and it got beyond control. The highest temperatures in the concrete buildings were reached in the west

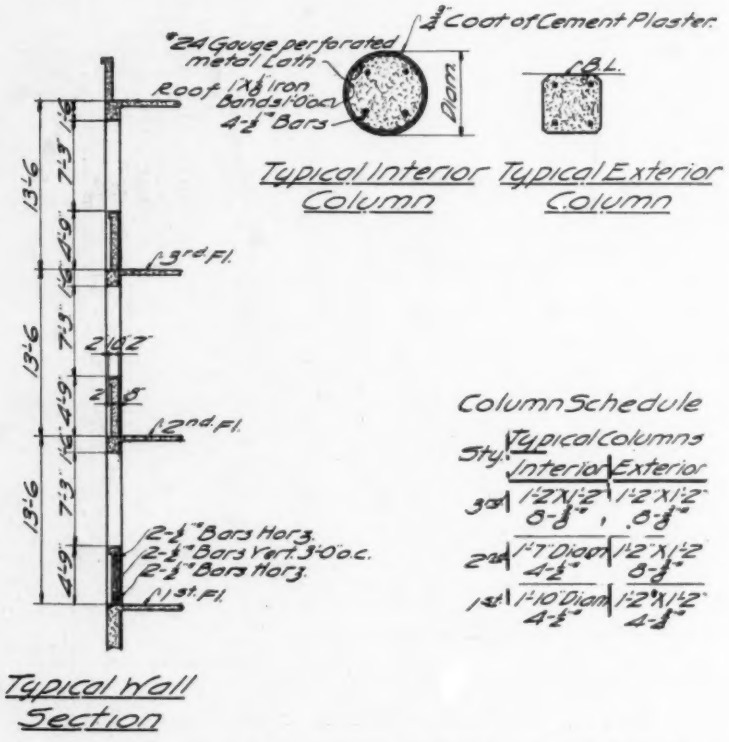
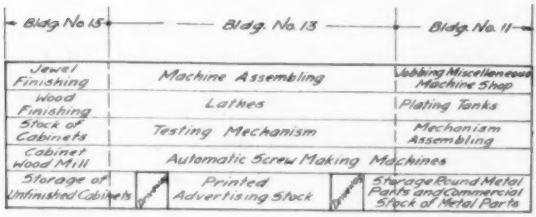
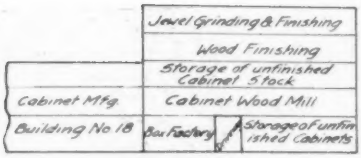


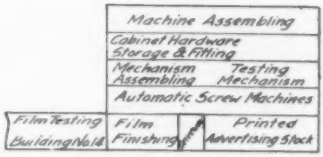

FIG. 14.—TYPICAL WALL AND COLUMN DETAILS OF BUILDING NO. 24.

end on the third floor of Building No. 24, where was stored a very large quantity of highly combustible material. In this building wherever a wood top floor was used it was in all cases entirely consumed, together with the wood sleepers embedded in cinder concrete. In the lower floor of Building No. 11, in which the interior columns showed the greatest damage, it is evident that extraordinarily high temperatures were obtained; in this

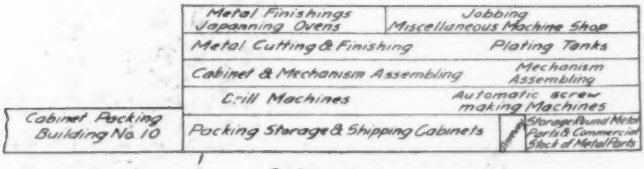


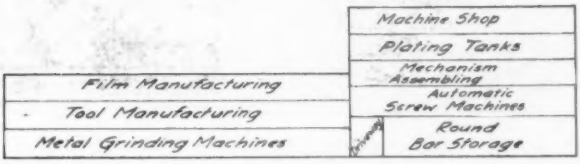

Building No 15



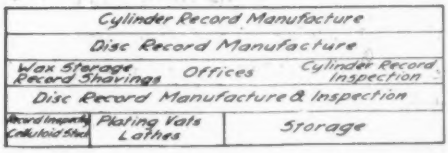

Building No. 13


Metal Finishings Japanning Ovens
Jobbing Miscellaneous Machine Shop
Metal Cutting & Finishing
Plating Tanks
Cabinet & Mechanism Assembling
Mechanism Assembling
Drill Machines
Automatic screw making Machines
Cabinet Packing Building No. 10
Packing Storage & Shipping Cabinets
Storage Round Metal Parts & Commercial Stock of Metal Parts

Building No. 11




Building No. 7




Building No. 24

Building No. 12

DIAGRAM 1.—LONGITUDAL SECTIONS OF BUILDINGS.

portion of the building where the damage was greatest the finished cabinet stock, piled to the ceiling, was entirely consumed.

The slagging of concrete during the fire in Building No. 12 was unquestionably due to the materials used in the process of manufacture at this point, and this is still under investigation.

The fused metal found in different parts of reinforced concrete buildings would seem to indicate that the fire reached an intensity of 1,000 deg. Fahr. in all these buildings, and in many

FIG. 15.—BUILDING NO. 11 AFTER THE COLLAPSE, SHOWING THE PROGRESS OF THE FIRE IN THIS BUILDING. THIS PHOTOGRAPH WAS TAKEN BETWEEN 9 AND 9.30.

cases as high as 2,000 deg. Fahr. In the third story of Building No. 24, the second and third stories of Building No. 15, and the first story of Building No. 11, temperatures in excess of 2,500 deg. Fahr. were reached.

### 7. BEHAVIOR OF THE CONCRETE IN THE FIRE.

In general, under ordinary conditions of fire exposure, the concrete, while cracked, was not damaged to any great extent

(Fig. 20). Where the fire was intense and of considerable duration, the damage to the concrete was very much greater. The variable effect of heat is shown very distinctly on the interior columns in the following way:

The columns in the office building where a small amount of combustible material was stored are little damaged. The same effect is found in the machine shop, where non-combustible occupancy caused lower temperatures. Where the fire was most intense, the interior column corners were spalled as described

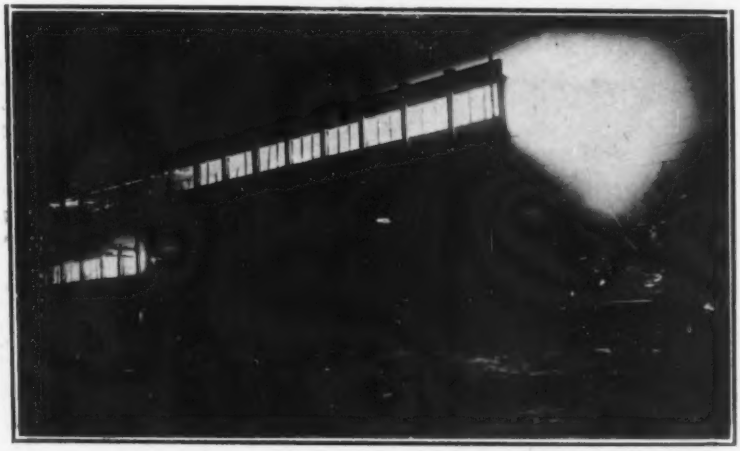


FIG. 16.—PROGRESS VIEW OF THE FIRE IN BUILDING NO. 24 ABOUT 8 O'CLOCK FROM WHICH IT WILL BE NOTED THAT THE FIRE WHICH HAD ENTERED THE BUILDING AT THE EAST END HAD PRACTICALLY TRAVERSED THE TOP FLOOR OF THIS BUILDING.

below. The damage to the concrete forming the columns (square in section) was very general in those stories where intense fire occurred. The concrete at the corners either spalled or cracked away from the center core of the column, which, with two exceptions, maintained its structural integrity (Fig. 21). This spalling or cracking at the corners varied in amount from 1 to 3 to 4 in., which generally left a column approximating an octagonal or circular section. In many cases the separation of the concrete at the corners of the column was along the line of the vertical reinforcement, but in very many cases the reinforcement was not

exposed. In this connection, it should be noted that the piers supporting the carpenter shop, which were not reinforced, were similarly spalled at the corners. The end walls in the three (3) upper floors of Buildings Nos. 13 and 15 were of reinforced concrete 8 in. thick and extended above the roofs of the adjoining buildings, which were completely destroyed; while this was in the hottest part of the fire, these walls were practically undamaged and are an admirable demonstration of the value of concrete walls as a fire barrier (Fig. 22).

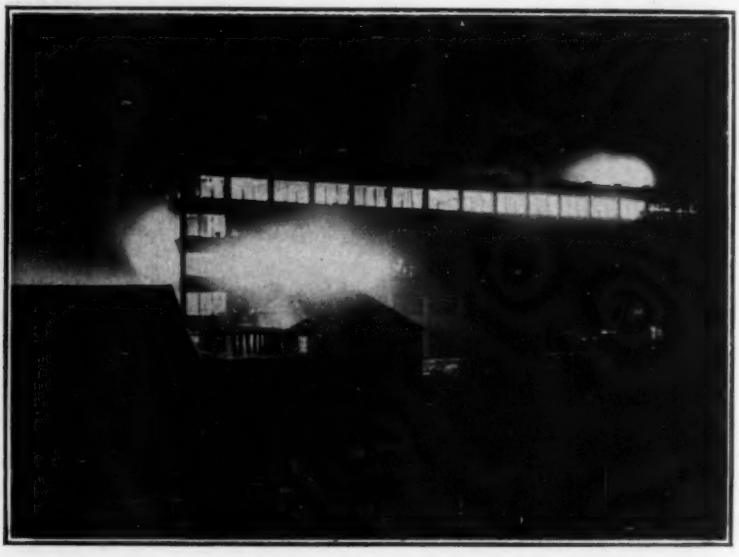


FIG. 17.—PROGRESS OF THE FIRE IN BUILDING NO. 24 AT ABOUT 8.30, AT WHICH TIME THE FIRE HAD NEARLY BURNED ITSELF OUT IN THE TOP FLOOR AND WAS MAKING RAPID HEADWAY IN THE EAST END OF THE THIRD FLOOR.

The behavior of the concrete under the unusual conditions of this fire fully demonstrated that it is an admirable material for fireproofing purposes, because of its very low heat conductivity and its toughness.

The intensity of the fire and the rapidity with which it traveled through the buildings subjected the concrete to extremely rapid expansion. Building stone, under similar conditions of rapid expansion, splinters and spalls, and while concrete is much

FIG. 18.—WEST END OF BUILDING NO. 24 AFTER THE FIRE HAD NEARLY BURNED OUT IN THE THREE UPPER STORIES.

tougher, it is subject to the same action. This action is proportional to the brittleness of the material. Under the conditions prevailing in a fire of usual severity there is an opportunity for concrete to adjust itself to increases in temperature and the danger of spalling is, therefore, greatly reduced.

There seems to be a general feeling that the damage to wall columns was the result of the application of water to the heated

FIG. 19.—BUILDING NO. 15 TAKEN BETWEEN 9 AND 9.30 OF THE FIRE IN THE TWO LOWER FLOORS.

concrete. The evidence available does not confirm this as a primary cause, as it is known that no water was played on some of the wall columns which were damaged.

The question of the expansion of structures from intense heat applied over large areas is one that deserves most careful consideration, and is a matter upon which very little information is available. Careful study is being made for the purpose of contributing information on this subject. The San Francisco fire showed, to a limited extent, the effect of expansion on struc-

tures, but most of the available information applies to relatively small units; the Edison fire is one of the first cases where an

FIG. 20.—INTERIOR VIEW OF THE SECOND FLOOR OF BUILDING NO. 25 (OFFICE BUILDING), SHOWING THE COMPLETE BURN OUT OF THE CONTENTS AND THE CONDITION OF THE UNDAMAGED CONCRETE STRUCTURE.

opportunity has been had to study the effect in long buildings under conditions of a conflagration in which extreme temperatures have developed. In a study of this character it is unfor-

tunate that comparison with some other type of construction could not have been included, but it is believed that concrete

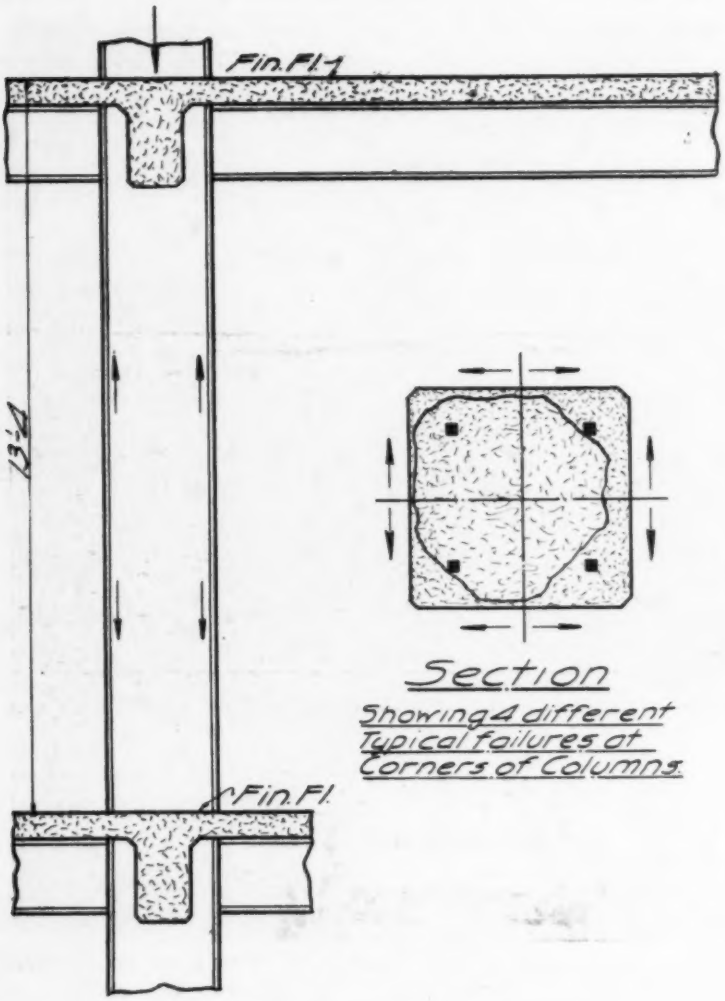



FIG. 21.—TYPICAL SPALLING OF INTERIOR COLUMNS.

stood these unusual conditions at least as well as could reasonably have been expected of any other material.

Many statements have been made as to the depth of cal-

cination of the concrete, but no one is competent to say whether the damage from this heat action extended any considerable distance below the surface of the concrete. At local points—for example, Building No. 12, where extraordinary conditions prevailed—the slagging of the concrete, which also carried with it the destruction of other materials, is not considered in this analysis. It will further be recognized that the large area of concrete buildings standing permitted rather an unusual opportunity for securing a number of examples where the damage had

FIG. 22.—BUILDINGS NOS. 11, 13, AND 15, SHOWING THE GENERALLY GOOD CONDITION AFTER THE FIRE, PARTICULAR ATTENTION BEING DIRECTED TO THE REINFORCED CONCRETE CURTAIN WALLS IN THE ENDS OF BUILDINGS NOS. 13 AND 15.

been apparently great, and in a fair analysis, due consideration should be given to the fact that in the greatest portion of these buildings the concrete remained firm and hard and intact after this severe heat treatment.

8. BEHAVIOR OF OTHER PARTS OF CONCRETE BUILDINGS.

The 3 and 4-in. plaster of Paris cinder blocks which were used in the interior partitions and for stair and elevator enclosures were all damaged, and under severe exposure were com-

pletely destroyed. Such tin-clad doors as had been installed were 1 to 1½ in. thick, and did not conform to the present requirements for proper fire doors, and in no case did they offer substantial resistance to the fire. The windows with wood frames and sash, glazed with plain glass, which were used throughout (with the exception of the monitors in Building No. 24 where metal frames and sash were used with plain ribbed glass), gave little resistance to the fire, the wood frames and sash being quickly consumed. The metal frames and sash in Buildings Nos.

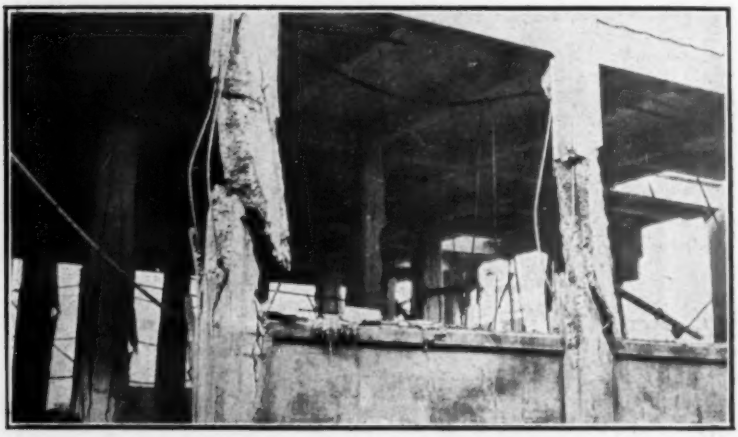


FIG. 23.—NORTHEAST CORNER, FIRST STORY OF BUILDING NO. 11.

11 and 24 were badly damaged, while the glass was, in nearly every instance, melted out of the sash. The wood topping on concrete floors, including the sleepers, was entirely consumed (Fig. 24).

### 9. DAMAGE TO CONCRETE IN EACH BUILDING.

The cracking of the concrete at the corners of interior columns was quite general and to a very limited extent occurred also at the underside of beams and girders. Figure 7 gives a summary of the extent of the damage to the concrete. Floor slabs were in general not damaged by the fire other than that small cracks were to be noted on the underside along the lines of the reinforcement; the concrete, however, seemed to be sound

and there was practically no spalling. The roof slabs generally expanded from the beams and girders along the ridge of the roof.

The damage to beams and girders was at the bottom and most severe along the edges, being quite general in the line of the reinforcement; the concrete usually remaind in plac and afforded protection to the reinforcement; on the sides it was practically undamaged.

It was noted that in the beams and girders and frequently in the columns expansion bolts or other metallic connections for

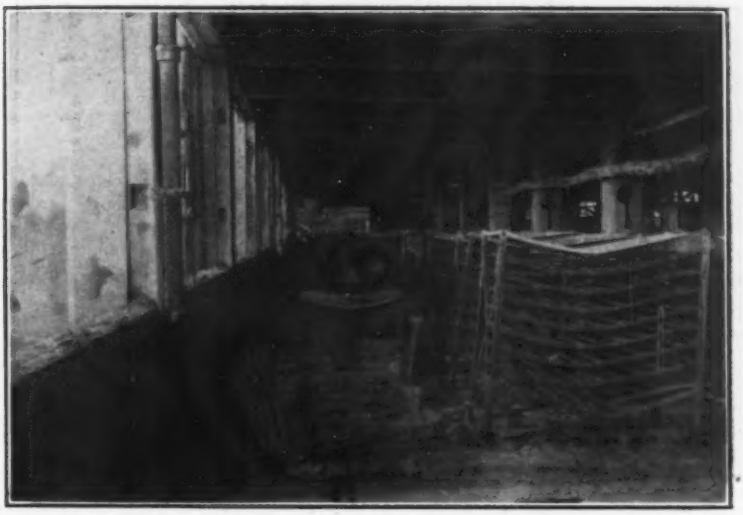


FIG. 24.—INTERIOR VIEW OF THE FOURTH FLOOR OF BUILDING NO. 24 AFTER THE FIRE. ALL THE COMBUSTIBLE MATERIAL IN THIS FLOOR WAS CONSUMED INCLUDING THE WINDOW SASH AND FRAMES, THE CONCRETE STRUCTURE BEING IN EXCELLENT CONDITION.

attaching piping, shafting, etc., to the structure had been embedded in the concrete in such a manner as to form a direct heat connection to the reinforcement (Fig. 25); and it is apparent that it is not good practice to embed metallic connections in such a way that they may become mediums for the transmission of heat into the interior.

The most serious damage to the reinforced concrete work was in the wall columns near the ends of the buildings. This is

especially true on Lakeside Avenue fronts of Buildings Nos. 11 and 15 (see Fig. 26 and 27); first story of the north end of Building No. 11 (see Fig. 28 and 29); and the third story both sides of Building No. 24. (The location of these columns is indicated on Fig. 7 and 12.) The broken portions of all these fractured wall columns indicate that there was a movement of the portion of the building above the fracture in a horizontal direction from the center of the building toward the ends, and the data

FIG. 25.—INTERIOR VIEW OF BUILDING NO. 11, SHOWING THE DAMAGE TO COLUMNS AND FLOOR BEAMS. ATTENTION IS DIRECTED TO THE BOLTS IN BEAMS FOR ATTACHING SHAFTING, ETC., AND THE SPALLING OF THE CONCRETE AT THESE POINTS.

obtained, showing the progress of the fire, indicates that the portion of the building above the fracture was heated prior to that below (Fig. 31). At witness of the failure of a section of Building No. 11 (Fig. 26) noted that first two wall columns failed, followed by failure of interior columns and general collapse of three upper floors. Further evidence of the horizontal movement of the portion of the building above the broken columns was the "V" shaped space where the curtain walls joined the wall columns, which was very pronounced near the ends of the buildings.

Measurements, taken after the concrete had cooled, from plumb lines dropped from the corners of the buildings indicated that the corner columns had returned nearly to their former position. The several wall columns which failed indicated that this was primarily due to the fact that the expansion of the building above was greater than that below the fracture; and since in all cases the distance from the center of the building to the point near which the columns were broken was greater than 100 ft. and the

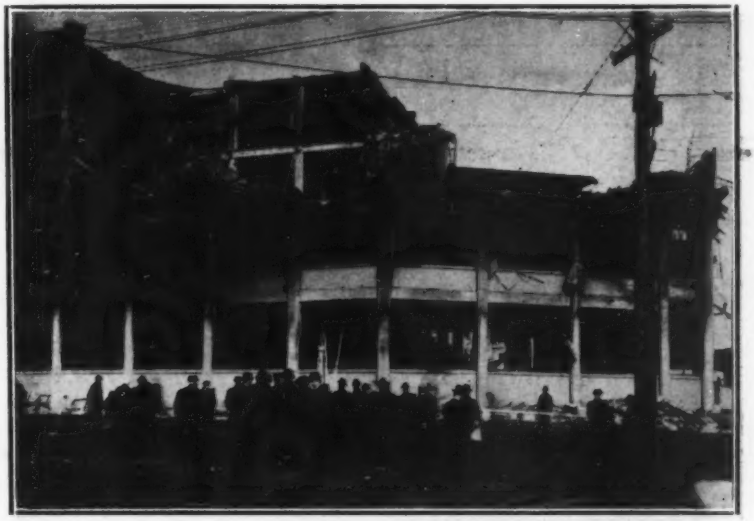


FIG. 26.—BROKEN WALL COLUMNS, 3D STORY AT BUILDING NO. 15. NO. 11 AFTER THE FIRE.

free expansion for this distance for a difference in temperature of 1,000 degrees would be over 6 in., it would seem likely that this expansion was a factor in the failure of these wall columns. This manner of failure in the wall columns is illustrated in Fig. 31. Attention is directed to the failure of the wall columns (vertically over each other in the first, second, and third floors) in the south side near the west end of Building No. 24. (See Fig. 32.) From the record of the progress of the fire the main failure probably occurred first in the third floor, followed by failure in the second floor, and later by failure in the first floor. The most remarkable feature, to be observed after the fire, was that the monolithic

construction was such as to transfer the load which these fractured wall columns had carried either to adjacent interior columns or adjacent wall columns, probably the former. In the former case the floor construction, with stiff connections to columns, acted as a cantilever above the broken columns to transmit the load back to the interior columns. In the latter case the wall construction acted as an arch or truss in transferring the load to the wall columns on either side of those broken. Where

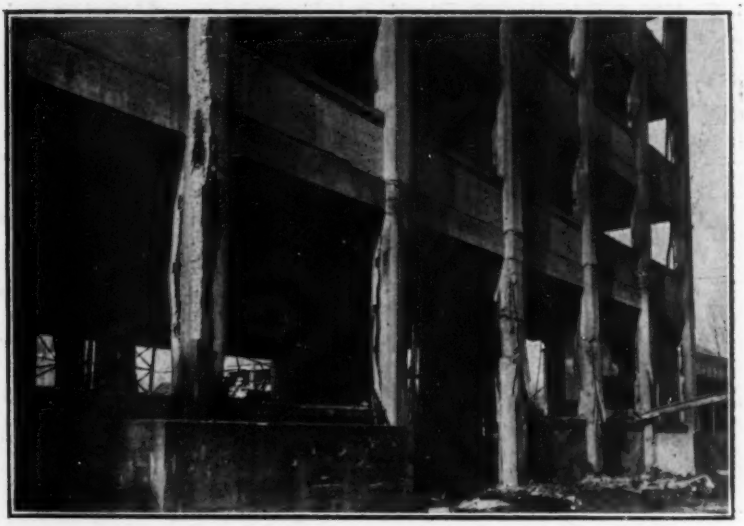


FIG. 28.—COLUMNS IN THE FIRST FLOOR OF BUILDING NO. 11 IMMEDIATELY UNDER THE PORTION OF THIS BUILDING SHOWN IN FIGURE 6, SHOWING THE MANNER IN WHICH THE COLUMNS WERE AFFECTED BY THE HEAT AND DEMONSTRATING THE REMARKABLE MONOLITHIC CHARACTER OF THE UNINJURED STRUCTURE THUS DEPRIVED OF THE SUPPORT OF THE WALL COLUMNS.

the wall columns had failed it was noted that generally the adjacent end columns had not. This is probably due to the lesser load on the end column and the absence of restraint by the curtain wall.

Interior columns of the reinforced concrete buildings were all square in section with the exception of those in Building No. 7, which are circular in the first and second stories. (Fig. 33.)

TABLE SHOWING NUMBER AND PER CENT. OF INJURED COLUMNS.

| Bldg. Floor. | Total No. of Cols. | No. Intact. | Cols. Badly Spalled. | Spalled to Reinforce. | Failed. | Per cent. Seriously Injured, including Failures. | Per cent. of Failures. |
|---|---|---|---|---|---|---|---|
| No. 24–1st | 73 | 66 | 3 | 2 | 2 | 9.6 | 2.7 |
| 2nd | 73 | 58 | 9 | 3 | 3 | 20.5 | 4.1 |
| 3rd | 73 | 43 | 13 | 7 | 10 | 41.1 | 13.7 |
| 4th | 73 | 63 | 6 | 0 | 4 | 13.7 | 5.5 |
| 5th | 73 | 43 | Interior Cols. on this Fl. were $\frac{3}{16}$ x 8-in. Pipe, Concrete Filled. All except 2 were deflected 1 to 3 in. | | | | |
| No. 11, 13, 15 | | | | | | | |
| 1st | 199 | 120 | 17 | 56 | 6 | 39.7 | 3.0 |
| 2nd | 199 | 160 | 18 | 18 | 3 | 19.6 | 1.5 |
| 3rd | 199 | 116 | 16 | 39 | 28 | 41.7 | 14.07 |
| 4th | 199 | 120 | 42 | 13 | 24 | 39.7 | 12.06 |
| 5th | 199 | 159 | 16 | .. | 24 | 20.1 | 12.06 |

Columns noted as intact include those having surface cracks not of a serious nature.

Columns noted as spalled include those where concrete is so separated from body of column by cracks as to be of no structural value even if the material has not actually fallen off.

Per cent. seriously injured does not include those that failed.

Columns classed as failed are those which have lost their ability to carry load

The above is a portion of the Table accompanying the National Fire Protection Association report of the Thomas A. Edison Inc. Plant fire.

Damage to the concrete at the corners of square columns was very general in those stories where fire occurred. Only two interior columns failed,—one in the first story of Building No. 11 near the south end of Building No. 7; this column is shown in Fig. 34, and it is to be noted that it is in line with the inside court

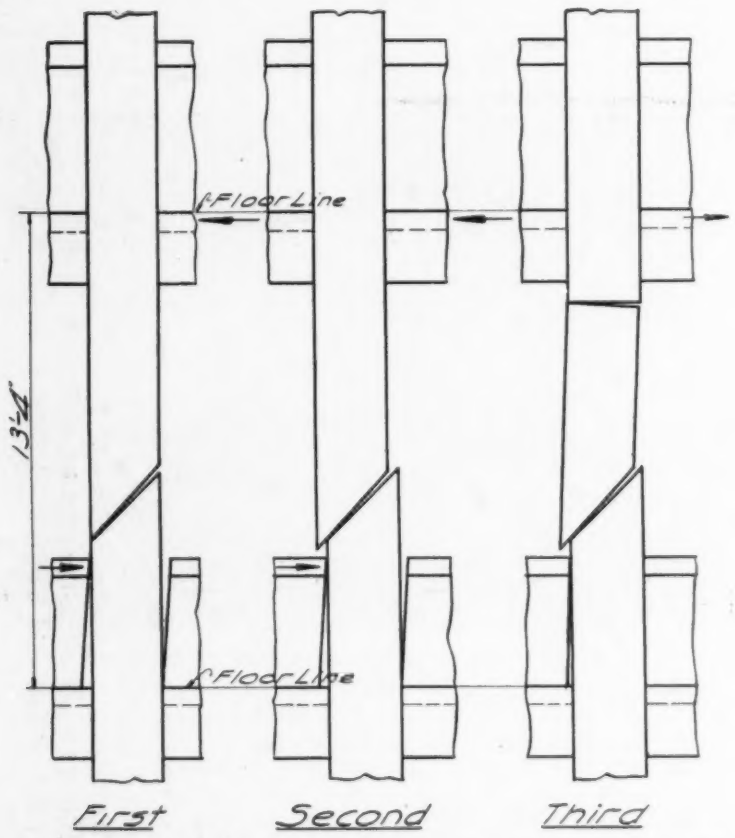



FIG. 31.—SKETCH SHOWING TYPICAL SECTION OF BROKEN WALL COLUMNS.

wall of the Lakeside Avenue wings of Buildings Nos. 11 and 13, and the primary cause of failure was probably similar to that for the other wall columns. The second interior column which failed is in the third story of Building No. 24 (Fig. 35), at which point there was an extremely hot fire, and in addition this column was

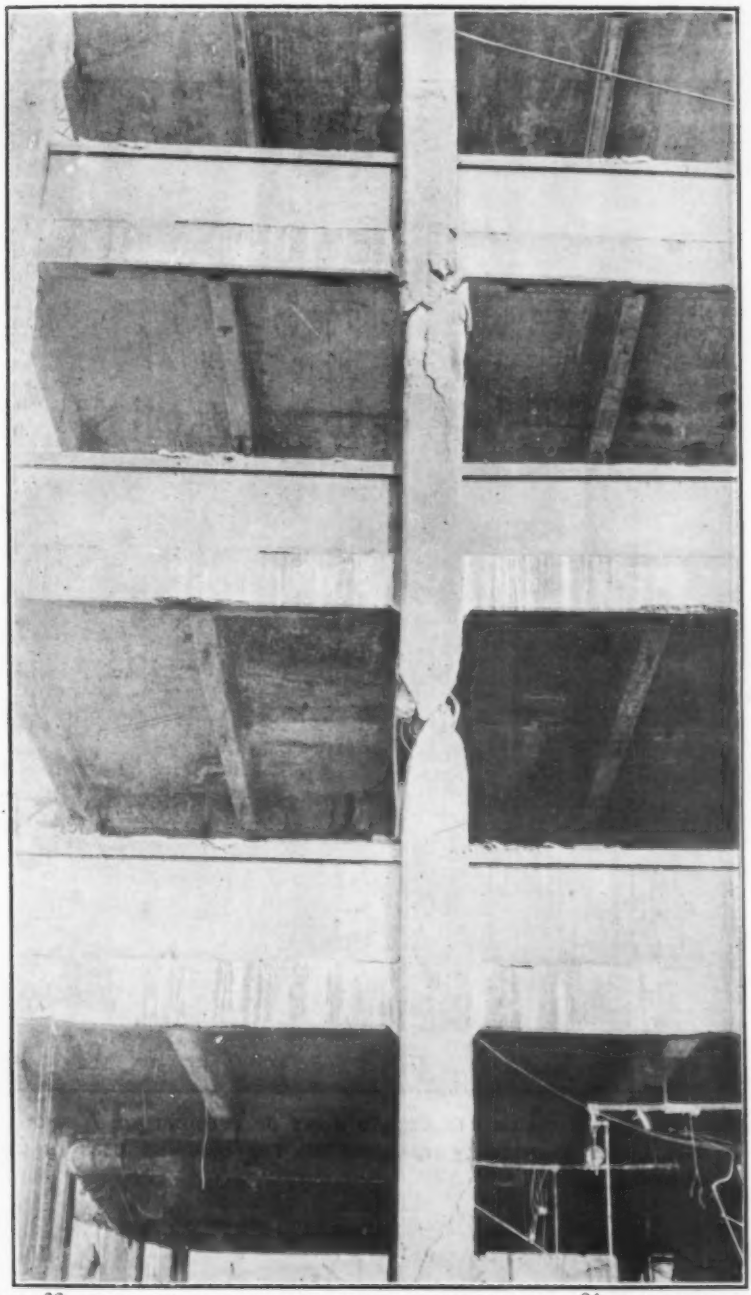


FIG. 32.—VIEW NEAR THE WEST END OF BUILDING NO. 24, SHOWING THE FAILURE OF COLUMNS IN THE SAME VERTICAL PLANE IN THE 1ST, 2D AND 3D STORIES.

FIG. 33.—INTERIOR COLUMN IN THE 2D STORY OF BUILDING NO. 7, SHOWING THE EFFECT OF HIGH TEMPERATURE ON THE COLUMNS USED IN THIS BUILDING.

overloaded by additional loads thrown upon it by failure of the adjacent wall columns. The general damage to interior columns

FIG. 34.—INTERIOR COLUMN IN FIRST STORY, BUILDING NO. 11, SHOWING THE EFFECT OF EXPANSION OF THE BUILDING. THE UNDAMAGED COLUMN SUPPORTING THE OTHER END OF THE BEAM SHOWN IN PICTURE SHOULD BE NOTED.

(Fig. 23) seems to have been primarily due to the fact that the surface of the concrete expanded much more rapidly than that

in the interior of the column. This expansion along the longitudinal surface of the column, in addition to the lateral expansion developed by the extremely rapid heating of the two faces at each corner, caused the spalling noted. Damage of the same character, although to a lesser degree, to the edges and the bottom of beams and girders also resulted under these conditions; although it should be noted that in the case of the beams and

FIG. 36.—COLUMNS SUPPORTING THE ROOF AT THE WEST END OF BUILDING NO. 24, SHOWING THE BROKEN FLOOR SLAB AT THE BASE OF THE CENTRAL COLUMN IN THE PICTURE. ATTENTION IS ALSO DIRECTED TO THE GROOVES OCCUPIED BY THE WOOD SLEEPERS WHICH HAVE BEEN ENTIRELY CONSUMED.

girders the concrete was not at the time in compression but in tension, and this probably explains the lesser damage.

There were two types of round columns in these buildings: The first was the Lally column, which supported the roof of Building No. 24. This column consists of a wrought iron pipe forming an outer shell which is filled with concrete. The intense heat in the floor of this building expanded the outer shell so rapidly that it blistered and flowed so as to form the annular bulge to be seen in Fig. 36. The utility of this shell for structural purposes was destroyed and the concrete core in most cases

carried the load; in some cases the column buckled as a result of the heat and failed.

The other type of column having a circular section was that in use in the two lower stories of Building No. 7. This column consists of an outer shell of punched metal lath covered with 1 in. of Partland cement plaster and of a core of reinforced concrete. While the actual strength of the column after the fire can only be determined from tests, it is evident that these columns maintained their structural integrity and successfully performed the function as supports for these floors, and further, gave evidence which indicates that a column of circular form is preferable to one of square section. While the first in this building was not as severe as in some of the other reinforced concrete buildings, it was noted that at the elevator shaft, where the fire conditions were alike in the two lower stories with circular columns and in the top story square columns, the corners of the square columns were damaged (similar to the damage to square columns in other buildings), while there was very little damage to the circular columns. The damage to circular columns consisted in the pitting and spalling of surface of the cement plaster.

There was an apparent lack of stiffness in the floors, which was undonbtedly due largely to the casting of the slabs and beams and girders separately, which was probably a contributing cause to the sagging in some of the floors. Reinforced concrete is most efficient as monolithic construction, and buildings of this character require, in order to develop the full strength of the structure, a positive bond between the slab and the floor frame.

### 10. EFFECT OF DESIGN ON THE PORTIONS DAMAGED BY THE FIRE.

There seems to be a general tendency to criticise the design of these reinforced concrete buildings, but it does not appear that this was a serious factor in the damage to the concrete; and it should be noted that most of the details which characterize this design were those in use at the time these buildings were constructed; that is, this design should not be criticised from the viewpoint of current practice, but rather from the practice at the time the buildings were designed. Comment is here made on the following points usually the subject of this criticism:

1. The stirrups used in the beams and girders consisted of two "L" sections lapped under the main reinforcement instead of the usual "U" section. This detail is not objectionable on account of its strength, because it gives a stronger anchorage at the bottom of the beam or girder than is usually obtained; but better anchorage to the slab should have been provided.

2. The vertical reinforcement in all columns was not provided with the usual metallic ties. If such ties had been used they probably would have held the concrete together, but would not have prevented the damage to the exterior shell of the column. After the heat reached the reinforcement it was absorbed faster than by the surrounding concrete, and the swelling of this metal, while not a primary cause of the damage to the concrete at the corners of columns, did result in serious stripping of the concrete from the columns in many places.

## 11. ESTIMATE OF COST TO RESTORE ALL REINFORCED CONCRETE WORK.

In view of the uncertainty as to the damage done and the repairs necessary, which can only be determined after load tests and a more complete investigation, it is impossible to state the exact cost of the restoration. The officials of the Thomas A. Edison, Inc., state that the salvage in machinery in the reinforced concrete buildings will be at least 94 per cent; of the buildings, 87 per cent, and the cost for the restoration of these buildings from 10 to 15 per cent.

Respectfully submitted,

CASS GILBERT, *Chairman*,
WALTER COOK,
WM. H. HAM,
RICHARD L. HUMPHREY,
RUDOLPH P. MILLER,
C. L. NORTON,
J. KNOX TAYLOR,
E. J. MOORE, *Secretary*.

# REPORT OF COMMITTEE ON REINFORCED CONCRETE HIGHWAY BRIDGES AND CULVERTS.

It has seemed best to the Committee to cover certain points in specifications for Reinforced Concrete Highway Bridges and Culverts by definite specifications, but certain other points must be varied according to the special circumstances of the case and, therefore, what is considered to be the best engineering practice is stated and recommended for adoption by the public. If any of the other national associations, of recognized standing, take up the same work, it would seem well to co-operate with them. It has been the policy of the Committee to adopt the standard specifications or recommended practice of any national society whose authority was generally admitted.

The following specifications and recommendations are submitted to the Institute, not as final, nor even as voicing the unanimous opinion of the members of the Committee, but merely as preliminary suggestions, in the hope of bringing out the discussion of doubtful points and information regarding questions which the Committee could not answer. Any information or criticism will be thankfully received and carefully considered.

## SPECIFICATIONS.

1. The specifications for *Reinforcement* and *Cement* should conform to those of the American Society for Testing Materials.

2. The specifications for *Aggregates* should conform to those of this Institute.

3. The *Unit Stresses* in reinforced concrete, and reinforcement, should conform to the recommended practice of this Institute.

## RECOMMENDED PRACTICE.

### DEAD LOADS.

4. The following are given as average weights per cu. ft. of the materials mentioned, but if the weights of the materials to be actually used are definitely known to be different from those given, the correct weights should be used.

| Materials | Lbs. per Cu. Ft. |
|---|---|
| Earth filling | 130 |
| Plain concrete | 150 |
| Reinforced concrete | 155 |
| Steel | 490 |
| Cast iron | 450 |
| Vitrified brick | 140 |
| Common brick | 125 |
| Granite and limestone masonry | 165 |
| Sandstone | 110 |
| Macadam-Telford | 150 |
| Pine, fir, etc | 42 |
| Oak and yellow pine | 48 |
| Creosoted | 60 |

5. The weight imposed by the earth filling, should be assumed to be equal to the weight of all the filling vertically above the portion of the structure in question, provided the depth of the fill does not exceed one half the total length of the bridge, from face to face of the abutments, in which case, the depth of the fill should be assumed to be equal to one half the length of the bridge, face to face of abutments. In the case of spandrel filled arches, if two planes inclined at an angle of 30° with the vertical one tangent to the arch extradus nearest each abutment, shall intersect below the surface of the road, only that part of the filling below said plane should be considered, otherwise the whole of the filling vertically above the arch should be assumed as resting upon it.*

This whole subject requires further investigation.

### LIVE LOADS.

*Class A Bridges*—Main thoroughfares leading from large towns.

6. In view of the extensive introduction of the *heavy motor trucks and traction engines*, and the probable general use of such vehicles in the future, it is recommended that bridges on main thoroughfares and other roads which are likely to be used for heavy hauling, be designed to carry 25-ton trucks, with axles about 10 ft. c. to c., 17 tons on rear axle and 8 tons on fore axle; wheels about 5 ft. c. to c. Outside of the large cities it is recommended

*This paragraph is not approved by Mr. Lemuel Holmes.

that such vehicles be assumed to be on the bridge at any one time, the likelihood of more than two being on the bridge, in a position to produce maximum stresses at the same time, is so remote that this assumption is considered safe. It is advised that such very heavy loads be considered as occupying only the ordinary width of road, about 8 ft. in width and about 35 ft. in length. Congested traffic of heavily loaded wagons or motor trucks will rarely impose a load of more than 100 lb. per sq. ft., over a considerable area. The above mentioned 25-ton truck gives a load of about 180 lb. per sq. ft., on the area actually occupied, but it is considered extravagant to assume that a large bridge is covered with such heavy loads. 125 lb. per sq. ft. is thought ample to assume for the loading of spans more than 60 ft. long, in designing the trusses or main girders. It is thought to be safe to reduce this assumed load in the case of longer spans, to the following amounts:

| Length of Span, ft. | Assumed Load, lb. per sq. ft. |
|---|---|
| 80 | 110 |
| 100 | 100 |
| 125 | 90 |
| 150 | 85 |
| 200 and over | 80 |

With all intermediate spans in proportion.

7. *Sidewalk Loads.*—The greatest load that is liable to be imposed on a bridge sidewalk, occurs when there is some excitement in the neighborhood, which attracts a large crowd, and for which the bridge affords an especially good point of view. In that case the crowd forms a compact mass, against the railing, not more than 4 ft. deep, making a load seldom exceeding 90 lb. per sq. ft., over a very considerable space. The remaining portion of the sidewalk may be covered by a moving crowd which can scarcely weigh more than 40 lb. per sq. ft. It may be advisable, sometimes, to so design sidewalk slabs, that if a street car or motor truck accidently gets upon the sidewalk, it will not go through. Such accidents are so rare, that it is thought safe to allow materials to be stressed nearly to the elastic limit in such cases.

8. *Class "B" Bridges.*—Although it is impossible to determine beforehand, especially in the newer parts of the country, whether any given road is to be used for heavy traffic, it seems extravagant,

at least in the cases of larger spans, to design bridges to carry much heavier loads than can be expected to come upon them. It is recommended that bridges of this class be designed to carry 17-ton trucks, with axles 10 ft. apart, 6 tons on the front and 11 tons on the rear axle. This will allow for a considerable overloading of existing motor trucks. It is further recommended, that only one truck be assumed to be on the bridge at one time, in designing the floor system, that it be assumed to cover a width of 8 ft. and a length of 35 ft. and that the remainder of the bridge is covered with a load of about 100 lb. per sq. ft., for spans up to 60 ft.

9. The longer spans, the trusses and main girders should be designed for the following loads:

| Length of Span ft. | Assumed Load lb. per sq. ft. |
|---|---|
| 80 | 90 |
| 100 | 80 |
| 125 | 75 |
| 150 | 65 |
| 200 and over | 60 |

With intermediate spans in proportion.

10. *Sidewalks.*—Sidewalks should be designed to carry the same loads as in the case of Class "A" Bridges.

*Special Bridges.*

11. City bridges and bridges carrying traffic connected with mines, quarries, lumber regions, mills, manufactories, etc., require special consideration and should, of course, be designed to carry any load which can reasonably be expected to pass over them, bearing in mind the likelihood of heavy traction engines and motor trucks coming into extensive use in the not distant future.

*Stringer Loading.*

12. The maximum stress in a stringer, due to a wheel load, occurs evidently when the wheel is directly over it. It is not thought proper to assume any distribution of the load to adjacent stringers, unless the bottom reinforcement in the slab is made continuous. In that case the distribution is proportional to the relative stiffness of the slab and the stringers, said stiffnesses

being proportional to the moments of inertia, multiplied by the modulus of elasticity of materials and inversely proportional to the cube of the span. In determining this distribution, due account must be taken of the fact that deflection of the slab decreases toward the end of the stringers, and also of the fact, that whatever load is carried to the adjacent stringers, deflects them also. It is therefore recommended that wherever practicable the bottom reinforcement of slabs be made continuous over the stringers.

*Slab Loading.*

13. The distribution in a direction parallel to the supports of a concentrated load resting on a slab, supported at two opposite edges only, evidently depends upon the same principles as those mentioned under *Stringer Loading.* The main difference being that what corresponds to the stringer in the former case is of indefinite width in the present case. Adequate theoretical investigations of this question appear to be lacking. Some excellent experimental work has been done but not of sufficient extent to really prove much. Your Committee hopes to make a more complete report on this subject later. For the present it seems fair to assume that the distribution each side of a concentrated load is equal to about one-third the length of the span, and that the cross reinforcement should be designed accordingly, which would require it to have an area of at least one-half of the principal reinforcement. The distribution of a concentrated load through earth filling on the top of a slab, does not appear to be very well understood, the Committee has not, thus far, been able to find any records of adequate theoretical or experimental investigation of the subject. It is hoped to obtain more data later.

*Bridges Carrying Electric Cars.*

14. Electric traction is still in its infancy and nobody is able to forecast its future development. It seems probable, however, that it will not be profitable to run cars weighing more than 50 tons each, at a speed that would be permitted on any public road. If very high speeds are desired, the traction company will doubtless be required to operate over its own right-of-way. It is recommended that bridges carrying either urban or inter-urban electric cars be designed to carry 50-ton cars on two trucks,

at least in the cases of larger spans, to design bridges to carry much heavier loads than can be expected to come upon them. It is recommended that bridges of this class be designed to carry 17-ton trucks, with axles 10 ft. apart, 6 tons on the front and 11 tons on the rear axle. This will allow for a considerable overloading of existing motor trucks. It is further recommended, that only one truck be assumed to be on the bridge at one time, in designing the floor system, that it be assumed to cover a width of 8 ft. and a length of 35 ft. and that the remainder of the bridge is covered with a load of about 100 lb. per sq. ft., for spans up to 60 ft.

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spaced 20 ft. c. to c., each truck having two axles spaced 7 ft. c. to c. The Committee sees no reason for changing the customary practice of assuming that an axle load is distributed over 3 ties.

*Loading on Arches.*

15. The deflection of an arch being much less than that of a beam of the same length, the method recommended for determining the lateral distribution of a concentrated load over arch sheeting must be modified. It seems doubtful if the distribution in each direction can be greater than the thickness of the arch sheeting. This question should be investigated.

*Stresses in Arches.*

16. As all arches that are not provided with hinges, act as elastic arches until cracks are formed, due to excessive tension at some point or points in the concrete, it is manifestly proper to calculate the stresses in them according to the elastic theory. There is of course, no objection to using short methods in calculation or empirical rules, provided they give substantially the same results as the correct method, which is quite laborious.

17. As concrete is a very poor conductor of heat, it is not thought necessary in calculating reinforced concrete arches, to assume so much variation in temperature as is usual in designing steel structures, although the outside layers of concrete are of nearly the same temperature as the surrounding air, and those layers are stressed more heavily than any of the others, it is thought that an extreme variation of about 90 deg. in the Northern States is sufficient to allow for, in any case, and that can be reduced if the arch ring is thick or if there is much earth filling in the spandrels.

BEARING POWER OF PILES.

18. The usual formula for the safe bearing power of wooden piles is:

$$B=\frac{2WH}{S+1}$$

in which, $W$ = Weight of hammer in pounds,

$H$ = Height of fall of hammer in feet,

$S$ = Penetration in inches per blow, average of last few blows.

Recent extensive tests on the bearing power of piles indicate that if this formula is used it may be with the understanding that it gives a factor of safety of 2, as regards the load which may cause a settlement of $\frac{1}{4}$ inch.

19. If reinforced concrete piles are molded before being driven, the head of the pile is usually cushioned more or less, and the pile is generally much heavier in proportion to the weight of the hammer than is the case with wooden piles. It is thought that a reduction of about 25 per cent should be made in the above formula unless a very heavy hammer is used.

20. If concrete piles are molded in place, measures should be taken to prevent damage to them by the driving of neighboring piles or otherwise.

21. The Committee hopes to make some further investigations in the subject before submitting a final report.

## BEARING POWER OF SOILS.

22. This subject is under an investigation by a committee from the American Society of Civil Engineers. It may be some years before their final report is submitted and even that will be subject to revision from time to time, as human knowledge is extended. In the meantime the Committee would submit the following preliminary table:—

| Material | Safe Bearing Power, Tons Per Sq. Ft. |
|---|---|
| Quicksands and wet soils | 0.1 to 1.0 |
| Dry earth, according to depth below surface | 1 to 3 |
| Moderately dry clay confined | 2 to 4 |
| Dry stiff clay | 4 to 6 |
| Sand confined | 2 to 6 |
| Sand compact and cemented | 4 to 8 |
| Gravel cemented | 8 to 12 |
| Rock | 25 to 200 |

Foundations should be carried down below frost unless they are on rock and thoroughly drained. Soil that contains the roots of plants is generally compressible. Undisturbed soil is much less compressible than filling or similar soil, even though it has been in place many years. The bearing power of sand, gravel and dry

clay increases rapidly with the depth below the surface of the ground.

24. The pressure of earth against abutments, wingwalls and retaining walls varies so widely with the character and condition of the earth in question, that nothing more than a few general suggestions can be given. It is hoped that the Committee from the Society of Civil Engineers, which is studying the bearing power of soils, will take up this subject also. In ordinary cases where the filling is well drained its pressure will seldom be more than that of a liquid weighing 25 lb. per cu. ft., and is frequently much less. The pressure due to land slides is often several times this amount. If the filling is clayey and is allowed to become waterlogged, its lateral pressure is greatly increased. The effect of the freezing of the filling must also be considered.

### REQUIRED WATERWAY.

25. The usual formula for waterway for culverts is:

$a = CA^{\frac{3}{4}}$

in which,

$a$ = Required area of culvert in sq. ft.

$A$ = Drainage area in acres,

$C$ = a constant, depending on the length and character of the drainage area and may vary from about .3 to 2.0 in regions where the mean annual rainfall is about 50 in.

The capacity of the culvert will be much greater if the wingwalls are flush with the abutments and flare about 30 deg., the sides and bottom of the culvert are smooth and straight, and the bottom has a good slope.

If reliable information covering a number of years can be obtained regarding the adequacy of the old bridge or of another bridge crossing the same stream, it is much more useful in determining the size of a culvert or bridge than any formula.

Respectfully submitted by the Committee,

WILLIS WHITED, *Chairman.*
LEMUEL HOLMES.
A. N. JOHNSON.
A. M. LOVIS.
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